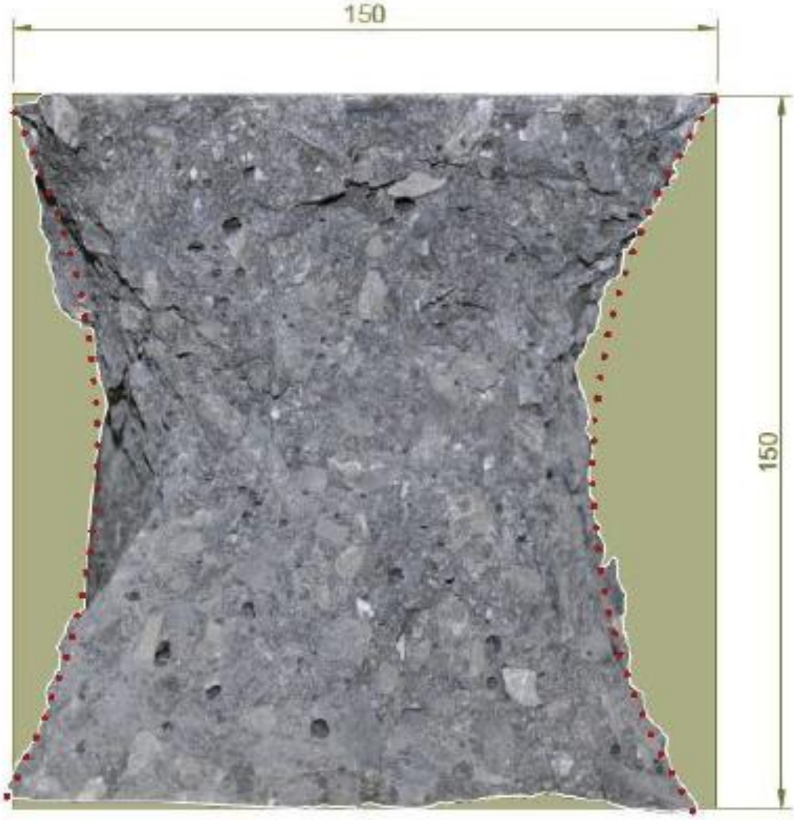
CONCRETE
STUDIES

SANIN **DŽIDIĆ** | ILDA **KOVAČEVIĆ** | SABINA **KOZLICA**



SARAJEVO, 2018

Sanin Džidić

Ilda Kovačević

Sabina Kozlica

CONCRETE STUDIES

Sarajevo, 2018

Authors:

Sanin Džidić
Ilda Kovačević
Sabina Kozlica

Publisher:

International BURCH University Sarajevo

Critical Review:

Assoc. Prof. Dr. Amir Čaušević, University of Sarajevo, Faculty of Architecture
Assoc. Prof. Dr. Merima Šahinagić-Isović, “Džemal Bijedić” University of Mostar, Faculty of Civil Engineering

Proofreading:

Dijana Misaljević, MA

Desktop publishing:

Authors

Date and Place:

June 2018, Sarajevo, Bosnia and Herzegovina

CIP - Katalogizacija u publikaciji
Nacionalna i univerzitetska biblioteka Bosne i Hercegovine, Sarajevo

624.012.4

DŽIDIĆ, Sanin
Concrete studies [Elektronski izvor] / Sanin Džidić, Ilda Kovačević, Sabina Kozlica. - El. knjiga. - Sarajevo : International Burch University, 2018. - 211 str.

Način dostupa (URL): http://www.eprints.ibu.edu.ba/3731/. - Nasl. s nasl. ekrana. - Izvor opisan dana 11.6.2018. - Bibliografija uz poglavlja.

ISBN 978-9958-834-60-8
1. Kovačević, Ilda 2. Kozlica, Sabina
COBISS.BH-ID 25822726

TABLE OF CONTENT

PREFACE

We love concrete! We love it because it is concrete. We love it because it is reliable. We love it for it is steady and durable. We love it because it is mysterious. We would like to discover some of its mysteries and share them with you!

That is why the "Concrete Studies"! Actually, this is a continuation of "Concrete Studies 2015-2016" and hopefully predecessor of some other future concrete studies. Again, we present you with the additional three studies conducted at the International BURCH University Sarajevo and University of Bihać in Bosnia and Herzegovina.

The first study is "Comparison of Fire Resistance of RC Slabs Determined according to Different Methods". We construct our buildings and facilities to last 50, 100 or even 200 years. There is a high probability that fire will eventually occur during the service life of practically every structure. Fire can happen anytime and anywhere. This study explores fire resistance of RC slabs that are the most sensitive concrete elements in fire situation. We compared the results of determination of fire resistance according to four different methods. We got some conclusions, but also opened an area for new research on fire resistance of some other concrete elements.

Concrete is principal construction material in Bosnia and Herzegovina. It is very common. However, concrete of high compression strength is almost unknown in Bosnia and Herzegovina. The study of "High-Strength Concrete (HSC) and Possibilities for Production in Bosnia and Herzegovina" discusses the recent history, advantages and disadvantages, application and benefits of it, as well as the constituent materials, mix design and proportioning and properties of high strength concrete. Experimental part of this study proves that it is feasible to produce high-strength concrete of slightly modified ordinary concrete mix improved by domestic admixtures and additives at minimal cost. This study aims to encourage concrete factories to produce it and engineers to apply it in their designs and actual construction.

In his book "Advanced Concrete Technology"[1], Dr. Zongjin Li said: "Fresh concrete requires considerable care, just like a baby." With 28 days of age, we consider concrete to be mature. However, to get concrete quality as required by design after 28 days, the curing procedure requires a whole set of steps, controls and tests. Unfortunately, concrete quality control in Bosnia and Herzegovina is usually related just to testing of concrete compression strength. The study of "Concrete Quality

---

[1] Li, Zongjin, 2011, "Advanced Concrete Technology", (John Wiley & Sons, Inc., Hoboken, New Jersey, USA ISBN 978-0-470-90239-4 (ebk),

Control according to European Standards –Case Study– Construction of the Waste Treatment Plant in Bihać" presents a unique example of Quality Assurance Program for construction project in Bosnia and Herzegovina, but with particular attention to concrete works in details. Testing of fresh concrete, testing of hardened concrete, and testing of steel reinforcement were integral parts of the QA Program during the implementation of the project. The approach presented in this study and implemented in the actual project could serve as a model, or at least for concrete quality control according to European Standards for other large construction projects in Bosnia and Herzegovina. Lessons learned from this project are important and experiences are tremendous.

We believe that engineers, architects, designers, construction and project managers, contractors, clients and students can gain and find useful some of our experiences based upon the critical approach and thinking, and also keeping in mind that knowledge, same as seed, cannot be just transplanted anywhere without a detailed analysis of each piece of the land, climate, and many other factors - and in this case any individual project. We do hope that findings from these studies can serve as a reference for your future endeavors.

We use this opportunity to thank our reviewers for their remarks, recommendations and suggestions. We'd also like to thank Ms. Dijana Misaljević for English proofreading that made this text better and Mr. Elmir Halebić for the design of the book cover. We also appreciate any effort and support by everyone who has in any way contributed to the process of publishing this book.

Authors

STUDY 1

COMPARISON OF FIRE RESISTANCE OF RC SLABS DETERMINED ACCORDING TO DIFFERENT METHODS

Sanin Džidić

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

A fire is an uncontrolled burning of fuel that expands in time and space, inflicting great losses, and putting people's lives at risk. Historically, fires were considered a great danger. Statistics show that the recorded number of structure fires, number of people injured, and number of deaths has significantly decreased in the last forty years. The reasons for this include greater public awareness, application of active and passive fire protection measures, as well as the fire protection regulations and standardization, fire prevention and science development. Despite modern firefighting methods and new technologies, fires still pose a great danger to both population and property.

Today, we design and build structures which can last for 50, 100 or even 200 years. There is a high probability that fire will eventually occur during the service life of practically every structure. Fire can happen anytime and anywhere.

Concrete elements are practically an integral part of every project and building. They have significantly higher fire resistance in comparison to steel or timber elements. However, reinforced concrete slabs are the most sensitive concrete elements to the effects of fire when compared to all the other reinforced concrete elements.

Therefore, this research focuses on determining the fire resistance of RC slabs using available world known methods. Methods used for determining fire resistance of slabs are EN 1992-1-2: 2004, Eurocode 2, Design of concrete structures, Part 1-2: Structural fire design (Tabulated Data and Simplified Method for Beams and Slabs), BRANZ Technical recommendation No. 8 – Method for Fire Engineering Design of Structural Concrete Beams and Floor Systems and ACI/TMS 216.1 – Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies.

A fire action to RC slabs is modeled using standard fires ISO 834-1 and ASTM E 119 E, depending on the method used for fire resistance determination.

This research considers aspects and methods for determining fire resistance of simply supported RC slabs of different spans (3, 5 and 7 m) and different depths (12, 15 and 17 cm) with variations of concrete cover ranging from 0.5 to 3 cm. Slabs were previously designed according to the EN 1992-1-1, Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings, with permanent action consisting of slab self-weight and flooring of 1.5 kN/m^2 and variable action of 2 kN/m^2.

All slabs were reinforced by welded ribbed meshes made of steel grade B500A, Ductility Class A, Yield = Re 500 MPa, or by straight ribbed bars made of steel grades

B500A or St-500-b.

Detailed data on RC slab designs and fire resistance periods are given in *Tables 4 - 12*.

FIRE MODELLING

When a serious scientific approach to fire resistance analysis and specifics of a fire was developed, there was a need to define the development of fire in test furnaces for conducting fire resistance testing of structures and elements. Based on detailed analysis, it has been concluded that most fires can be modeled through the curve of temperature-time dependence. The International Standard ISO 834-1, Fire Resistance Tests - Elements of Building Construction from 1999, is internationally accepted in Europe, Australia, New Zealand, and other countries, and it defines the standard temperature – time curve for fire modeling or exposure of test samples in test furnaces.

This standard is also accepted as BAS ISO 834-1 in Bosnia and Herzegovina [28] by the Institute for Standardization of Bosnia and Herzegovina, based upon the proposal of its Technical Committee BAS TC 37 – Fire Safety in Buildings. The curve is defined as:

$$T = 345\, log_{10}(8t + 1) + 20 \qquad (1)$$

where:

T - average temperature in the test furnace in ºC,
t - test time in minutes.

Figure 1 shows the Standard Temperature -Time Curve ISO-834-1 from 1999.

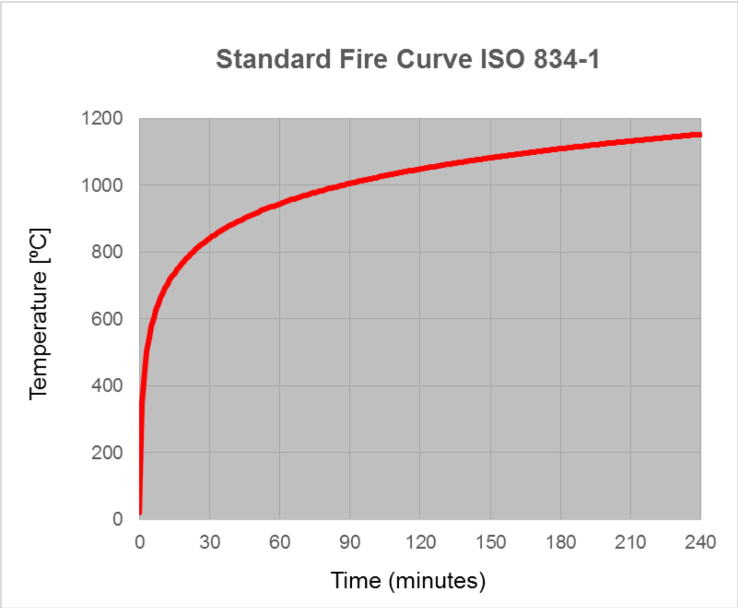


Figure 1 – Standard Temperature-Time Curve ISO 834-1

Table 1 displays values of average furnace temperatures according to the ISO 834-1 temperature-time curve for different test times.

Table 1 – Average Furnace Temperatures for Different Fire Test Times according to ISO 834-1

Time (t) [min]	**Average Temperature in Furnace T-T_0 [ºC]**
5	556
10	659
15	718
30	821
60	925
90	986
120	1029
180	1090
240	1133
360	1193

Eurocode 1 (EN 1991-1-2: 2002) also adopts this temperature-time curve as a nominal curve. In addition, Eurocode 1 also specifies the hydrocarbon and external fire curves as nominal curves for specific applications; they are not the subject of this research and they are not presented here.

The standard fire curve used in the USA is the Standard Fire Curve Temperature-Time ASTM E 119 [12]. It is presented with a number of discrete points as shown in the following table.

Table 2 – Temperature of Standard Fire ASTM E 119

Time (t) [min]	**Temperature of Standard Fire ASTM E 119 [ºC]**
5	538
10	704
30	843
60	927
120	1010
240	1093
480 and over	1260

Lie [12] gave several equations that mathematically approximate the ASTM E 119 curve, where the simplest one gives the temperature in function of time through the following relationship:

$$T = 750\left[1 - e^{-3.79553\sqrt{t_h}}\right] + 170.41\sqrt{t_h} + T_0 \tag{2}$$

where

t_h - test time expressed in hours.

Figure 2 illustrates the given relationship.

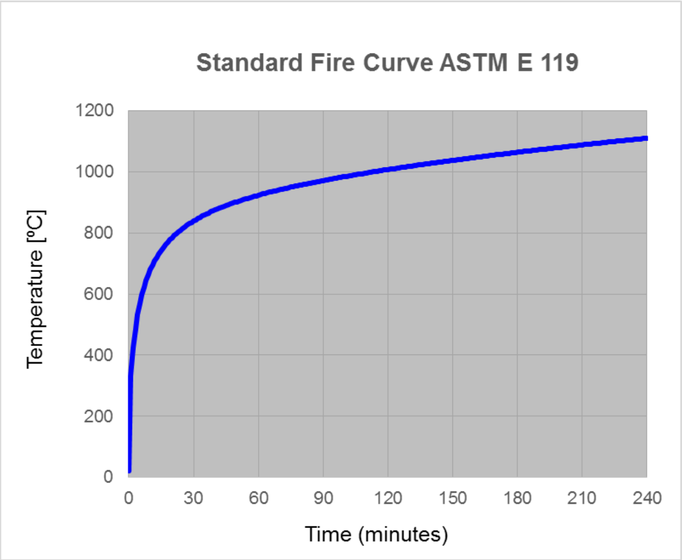


Figure 2 – Standard Temperature-Time Curve ASTM E 119

In order to correctly understand previous and subsequent considerations presented in this research, it is necessary to highlight basic aspects of a fire development in a room.

In general, while a fire starts small immediately after ignition, it has characteristics of an open fire. At the time of growth, it can be considered that the fire has a local character, and the average room temperature is relatively low. At this stage, effective control and fire retardation can be achieved by applying active fire protection measures through fire detection and fire detection systems, sprinklers and other systems designed to prevent and extinguish fires. After a certain time, and due to the increase in flue gases and radiation from the flame, the temperature in the room increases to several hundred degrees. When the temperature reaches a value of about 600 °C in the ceiling zone, flashover develops. After that, the fire comes to the stage of a fully developed fire, and the temperature reaches its maximum value, with the cancellation of active fire protection measures. When the temperature drops to the level of about 80% of maximal temperature, it is considered that the cooling phase has begun, because it has reached smaller amount of released heat and the fire enters its downward phase.

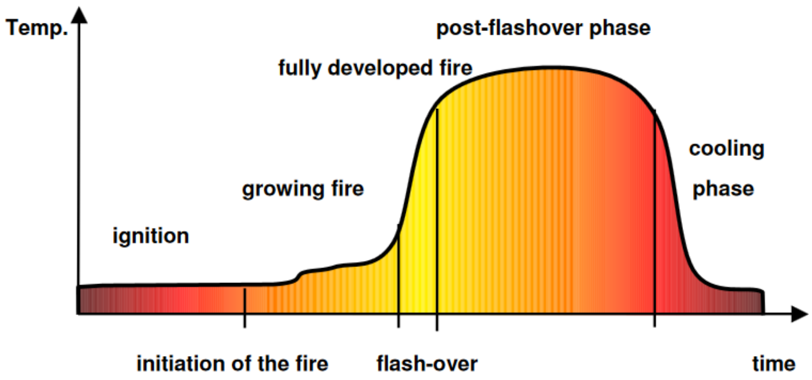


Figure 3 – Real Fire Model [11]

However, if we compare standard curve of fire development and real fire development, we can see that there is no decay phase at all in standard fire curves.

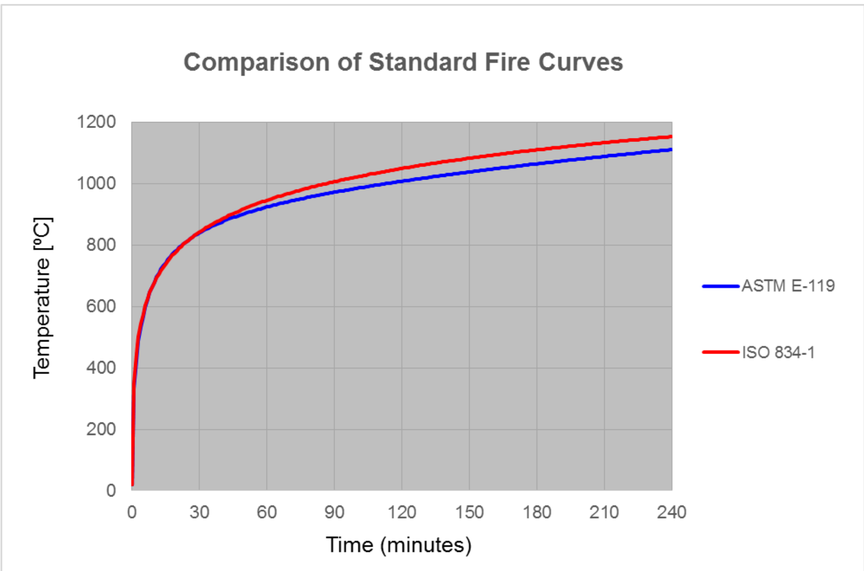


Figure 4 – Comparison of Standard Fire Curves

It can be noticed that standard fire curves do not correspond to any real fire situations; they exist only in controlled conditions of standard testing in test furnaces. A question of whether the test results obtained on structures and elements in such standard conditions apply to a real construction or element, is raised; however, this also opens up a whole new set of other questions to be discussed, which are not the focus of this research.

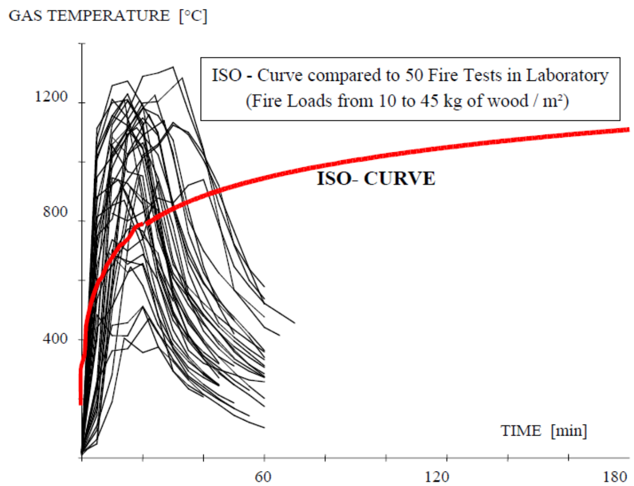


Figure 5 – Natural Fire Curves and ISO 834-1 Temperature–Fire Curve [15]

Parametric fire curves were developed based on these findings. In comparison to nominal fire curves, parametric fire curves provide more realistic assessments of the temperature in the fire compartment. Their basic purpose is to determine fire resistance of structural elements and structures. These curves take into account the size of the fire compartment, ventilation conditions, as well as thermal characteristics of the material applied at boundary surfaces of the observed fire compartment and other influence factors.

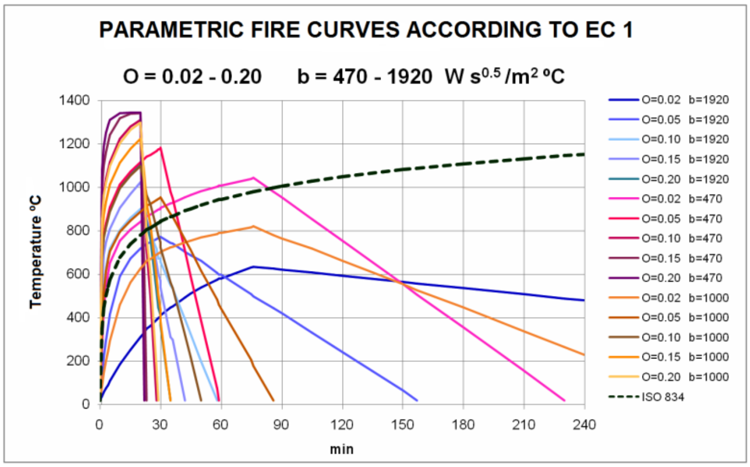


Figure 6 – Example of Parametric Fire Curves according to EC1 Developed for b = 470 -1920 J/m²s^1/2 ºC and O = 0.02 – 0.20 [6]

EN 1991-1-2:2002, Annex A, introduced parametric fire curves in standardization for the first time. It also specified limitations when dealing with these parametric fire curves. They can be applied only for fire compartments of up to 500 m^2 . The openings in the ceiling must be placed, and the maximum height of the fire compartment must not exceed 4 m. The limit values of the opening factor (O) and the coefficient b are shown in detail as follows.

Temperature-time parametric curves in the heating phase are given by:

$$\theta_g = 20 + 1325\left(1 - 0.324\, e^{-0.2t^*} - 0.204\, e^{-1.7t^*} - 0.472\, e^{-19t^*}\right) \quad [^{\circ}C] \qquad (3)$$

where

$$t^* = t \cdot \Gamma \qquad [h] \qquad (4)$$

$$\Gamma = \frac{(O\,/b)^2}{(0.04/1160)^2} \qquad (5)$$

$$b = \sqrt{\rho c \lambda}; \qquad 100 \le b \le 2200 \qquad [J/m^2 s^{1/2} K] \qquad (6)$$

$$O = \frac{A_v \sqrt{h_{eq}}}{A_t}; \quad 0.02 \le O \le 0.2 \qquad [m^{1/2}] \qquad (7)$$

with

ρ	- density of boundary of enclosure [kg/m^3];
c	- specific heat of boundary of enclosure [J/kgK];
λ	- thermal conductivity of boundary of enclosure [W/mK];
O	- opening factor [m$^{1/2}$];
A_v	- total area of vertical openings on all walls [m^2];
h_{eq}	- weighted average of window heights on all walls [m];
A_t	- total area of enclosure (walls, ceiling and floor, including openings) [m^2].

The maximum temperature of the heating phase t^* must be equal to t^*_{max}, where:

$$t^*_{max} = t_{max} \cdot \Gamma \qquad [h] \qquad (8)$$

$$t_{max} = max\left[(0.2 \cdot 10^{-3} \cdot \frac{q_{t,d}}{O}\ ;\ t_{lim}\right] \qquad [h] \qquad (9)$$

$$q_{t,d} = q_{f,d} \cdot \frac{A_f}{A_t} \qquad 50 \le q_{t,d} \le 100 \qquad [MJ/m^2] \qquad (10)$$

where

$q_{t,d}$ - is the design value of the fire load density related to the total surface area A_t of the enclosure. Following limits should be observed: $50 \le q_{t,d} \le$ 1000 [MJ/m2];

$q_{f,d}$ - is the design value of the fire load density related to the surface area A_f of the floor [MJ/m2] taken from Annex E of EN 1991-1-2:2002.

Based on the fire load, the amount of available energy can be determined. On the other hand, the temperature reached depends on the rate of the heat released. This phenomenon is called Rate of Heat Release or RHR, and it depends on the conditions of the ventilation of the fire compartment. Eurocode 1 predicts three fire growth rates: slow, where t_{lim} = 25 minutes, medium where t_{lim} = 20 minutes, and fast where t_{lim} = 15 minutes. Eurocode 1, Part 1-2, Annex E specifies RHR application depending on the occupancy of the fire compartment.

Temperature-time parametric fire curves in the cooling phase are given by:

$$\theta_g = \theta_{max} - 625\,(t^* - t^*_{max} \cdot x)\,; \qquad t^*_{max} \leq 0.5\,; \tag{11}$$
$$\theta_g = \theta_{max} - 250\,(3 - t^*_{max})(t^* - t^*_{max} \cdot x); \quad 0.5 < t^*_{max} < 2 \tag{12}$$
$$\theta_g = \theta_{max} - 250\,(t^* - t^*_{max} \cdot x); \quad t^*_{max} \geq 2 \tag{13}$$

Factor $x = 1$ if $t_{max} > t_{lim}$ and if $t_{max} = t_{lim}$, then $x = t_{lim} \cdot \Gamma/t^*_{max}$.

However, in further considerations, this research will use only standard fire curves ISO 834 - 1 and ASTM E 119.

EUROCODE 2: DESIGN OF CONCRETE STRUCTURES PART 1-2 – STRUCTURAL FIRE DESIGN

EN 1992-1-2:2004, Eurocode 2, Design of concrete structures, Part 1-2: Structural fire design, provides following three methods for determining the fire resistance of concrete structures:

- Tabulated data;
- Simplified calculation methods; and
- Advanced calculation methods.

This standard provides a choice between nominal and natural (parametric) fire curves in order to determine fire resistance of concrete elements and structures. The nominal (standard) fire is presented as a general curve, ISO 834 - for the purpose of classification and comparisons, but does not have any direct relationship with the characteristics of the building, its structure, or elements. Parametric fire curves elaborate calculation techniques based on the consideration of physical parameters that characterize the building or fire compartment. These fire curves were discussed earlier in this research.

Fire performance of concrete elements and structures exposed to the nominal fire should be observed through three basic criteria:

- Mechanical resistance, load bearing function (Criterion R);
- Integrity, separating function (Criterion E); and
- Insulation (Criterion I).

EN 1992-1-2:2004 defines that Criterion "R" is assumed to be satisfied when the load bearing function is maintained during the required time of fire exposure. The Criterion "E" (Integrity) is the ability of the separating element of the structure to prevent passing of flame and gases from one side to another side of the element when exposed to the standard fire from one side, thus preventing the occurrence of the flame on the unexposed side during the required time period. Criterion "I" may be assumed to be satisfied when the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K in consideration of the separating element of the structure when exposed to the standard fire from one side for the required time of fire exposure.

Required criteria for the observed element depend on the element’s function in the building. For example, when it comes to slab, it must meet all three criteria R, E and

I, while columns are usually expected to satisfy only the load-bearing function - Criterion R.

It is well known that the increase in temperature in concrete structures caused by fire reduces the compression strength of concrete and tensile strength of the reinforcement. Reduction of the characteristic compression strength of concrete is in the function of the temperature θ, using the reduction factor $k_c(\theta)$ allowing for decrease of concrete compression strength and is given in *Figure 7.*

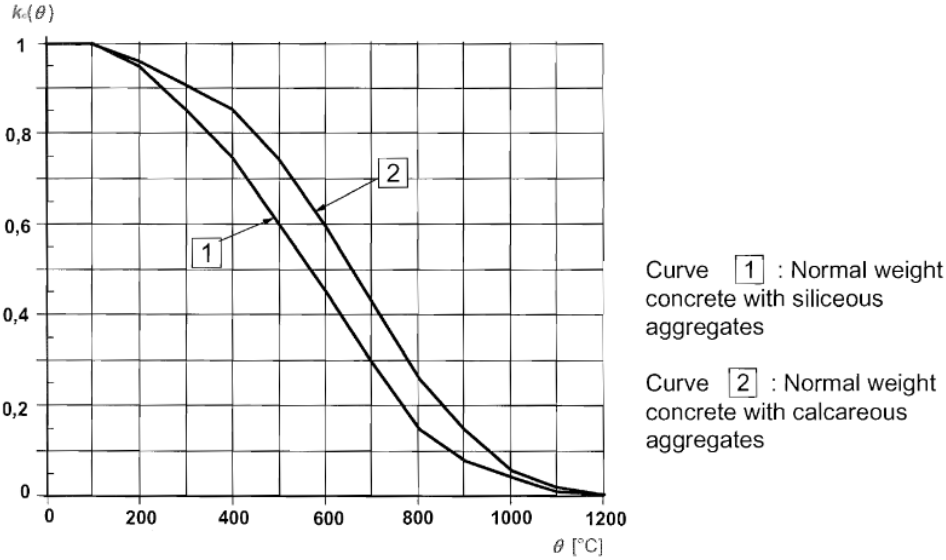


Figure 7 – Reduction Factor $k_c(\theta)$ Allowing for Decrease of Concrete Strength [28]

The reduction coefficient k_s (θ) allowing for decrease of characteristic strength (f_{yk}) of tension and compression steel reinforcement depends on the technological method of production of the steel reinforcement, as well as the stress and strain on elevated temperatures, which is shown in *Figures 8* and *9*.

When required for member analysis, the effect of fire on actions is accounted for by applying a reduction factor η_{fi} to the ambient design value:

$$E_{\mathrm{d,fi}} = \eta_{\mathrm{fi}} \cdot E_{\mathrm{d}} \tag{14}$$

where

- $E_{\mathrm{d,fi}}$ - design effect of actions in the fire situation;

- E_{d} - design effect of actions for the normal temperature design;

- η_{fi} -is the reduction factor for the design load level for the fire situation.

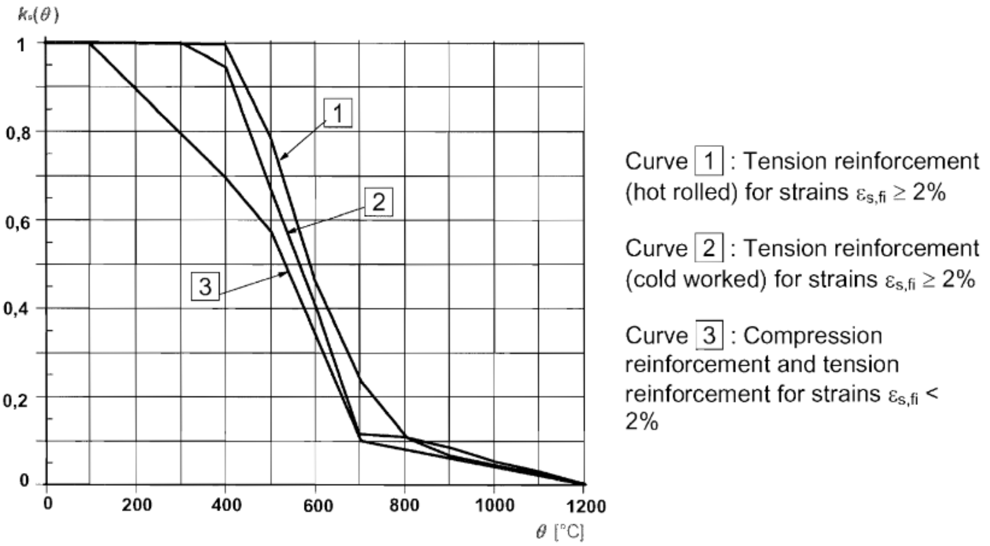


Figure 8 – Reduction Factor $k_s(\theta)$ Allowing for Decrease of Characteristic Strength f_{yk} of a Tension and Compression Reinforcement (Class N) [28]

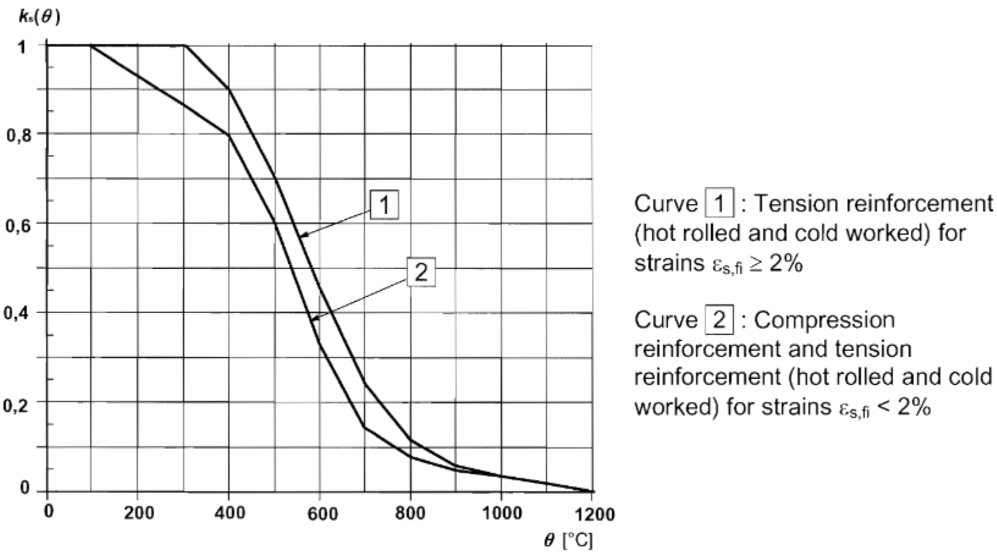


Figure 9 – Reduction Factor $k_s(\theta)$ Allowing for Decrease of Characteristic Strength f_{yk} of a Tension and Compression Reinforcement (Class X) [28]

Reduction factor η_{fi} for load combination, according to EN 1990 is determined as

$$\eta_{\mathrm{fi}} = \frac{\mathrm{G_k} + \psi_{\mathrm{fi}} \cdot \mathrm{Q_{k,1}}}{\gamma_\mathrm{G} \cdot \mathrm{G_k} + \gamma_{\mathrm{Q,1}} \cdot \mathrm{Q_{k,1}}} \tag{15}$$

or for the load combination as smaller value given by the following two expressions:

$$\eta_{fi} = \frac{G_k + \psi_{fi} \cdot Q_{k,1}}{\gamma_G \cdot G_k + \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1}} \tag{16}$$

$$\eta_{fi} = \frac{G_k + \psi_{fi} \cdot Q_{k,1}}{\xi \gamma_G \cdot G_k + \gamma_{Q,1} \cdot Q_{k,1}} \tag{17}$$

where

$Q_{k,1}$ - the principal variable load;
G_k - the characteristic value of a permanent action;
γ_G - the partial factor for a permanent action;
$\gamma_{Q,1}$ - the partial factor for variable action 1;
ψ_{fi} - the combination factor for frequent or quasi-permanent values given either by $\psi_{1,1}$ or $\psi_{2,1}$, see EN 1991-1-2;
ξ - reduction factor for unfavorable permanent action.

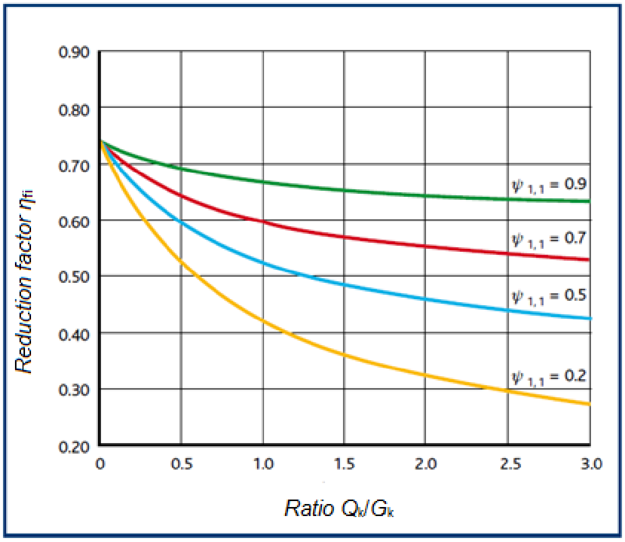


Figure 10 – Combination Factor $\psi_{1,1}$ [7]

EN 1992-1-2 recommends a value of η_{fi} = 0,7 as a simplification.

For application in structural fire design, values of partial safety factors for thermal and mechanical properties of materials (concrete and reinforcement) are taken $\gamma_{M,fi}$ = 1.0.

Tabulated Data

This method gives recognized design solutions for the standard fire exposure up to 240 minutes and applies to normal weight concrete (2000 to 2600 kg/m^3), made with siliceous aggregates.

The method is based on the determination of the axis distance of the reinforcement to the fire exposed side of the concrete element, and not on the thickness of the concrete cover, depending on the required period of fire resistance and the required function of the element. The following *Figure* defines the axial distance "a" from the centroid of the reinforcement to the fire exposed concrete face.

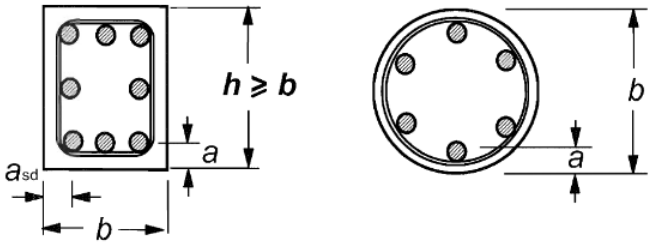


Figure 11 – Axis Distance [28]

If calcareous aggregates or lightweight aggregates are used in slabs, minimum dimensions of the cross-section given in tables may be reduced by 10 percent. Minimum requirements tabulated in tables from the aspect of dimensions of the cross-section and the axial distance required to meet Criterion R have their origin from the following equation:

$$E_{\mathrm{d,fi}}/R_{\mathrm{d,fi}} \leq 1{,}0 \tag{18}$$

where

$E_{\mathrm{d,fi}}$ - design effect of actions in the fire situation; and

$R_{\mathrm{d,fi}}$ - design load-bearing capacity (resistance) in the fire situation.

Table 3 – Minimum Dimensions and Axis Distance for Simply Supported Concrete Slabs [28]

Standard fire resistance	Minimum dimensions (mm)			
	slab thickness h_s (mm)	axis-distance a		
		one way	two way: $l_y/l_x \leq 1{,}5$	two way: $1{,}5 < l_y/l_x \leq 2$
1	2	3	4	5
REI 30	60	10*	10*	10*
REI 60	80	20	10*	15*
REI 90	100	30	15*	20
REI 120	120	40	20	25
REI 180	150	55	30	40
REI 240	175	65	40	50

l_x and l_y are the spans of a two-way slab (two directions at right angles) where l_y is the longer span.

For prestressed slabs the increase of axis distance according to 5.2(5) should be noted.

The axis distance *a* in Column 4 and 5 for two way slabs relate to slabs supported at all four edges. Otherwise, they should be treated as one-way spanning slab.

* Normally the cover required by EN 1992-1-1 will control.

Tabulated data are based on the reference load level $\eta_{fi} = 0.7$ unless otherwise stated. In order to ensure necessary axis distances in tensile zones of simply supported slabs, *Table 3*, Column 3 (one way), are based on a critical steel temperature of $\Theta_{cr} = 500$ °C. This assumption corresponds approximately to $E_{d,fi} = 0.7\ E_d$ and $\gamma_s = 1{,}15$, where E_d denotes the design effect of actions according to EN 1992-1-1. The minimum slab thickness h_s given in *Table 3* ensures adequate separating function (Criteria E and I).

Simplified Calculation Method for Beams and Slabs

The simplified calculation procedure for beams and slabs given in Eurocode 2, Part 1-2, Annex E, applies to beams and slabs, where the load is predominantly uniformly distributed and where the ambient temperature design is based on a linear analysis or on a linear analysis with limited redistribution.

This method is practically a continuation of the tabular method for beams exposed to the fire on three sides and on slabs in order to determine the bending capacity in situations where the axial distance is shorter to the bottom reinforcement than the one required in the tables.

Steel strength reduction factors used in this method are presented in *Figure 12*.

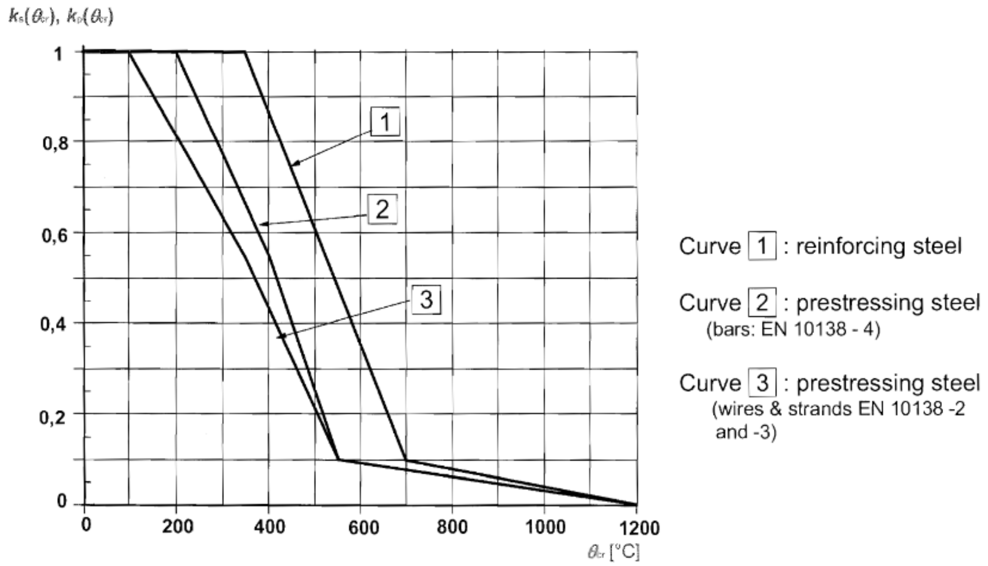


Figure 12 – Steel Strength Reduction Factors [28]

The main guideline of this procedure is to ensure that the design bending moment in fire conditions is less than or equal to the cross-sectional moment capacity in the fire situation:

$$M_{\mathrm{Ed,fi}} \leq M_{\mathrm{Rd,fi}} \quad (19)$$

In the focus of this research are simply supported reinforced concrete one-way slabs, thus, fire design moment for predominantly uniformly distributed load is:

$$M_{\mathrm{Ed,fi}} = w_{\mathrm{Ed,fi}} \cdot l_{\mathrm{eff}}^2/8 \quad (20)$$

where

$w_{\mathrm{Ed,fi}}$ - uniformly distributed load in the fire situation;

l_{eff} - effective length of the slab.

For the simply supported slab, the moment of resistance in the fire situation is determined according to the following relationship:

$$M_{\mathrm{Rd.fi}} = (\gamma_{\mathrm{s}}/\gamma_{\mathrm{s,fi}}) \cdot k_{\mathrm{s}}(\theta) \cdot M_{\mathrm{Ed}} \cdot (A_{\mathrm{s,prov}}/A_{\mathrm{s,req}}) \quad (21)$$

where

γ_{s} - the partial safety factor for steel used in EN 1992-1-1;

$\gamma_{\mathrm{s,fi}}$ - the partial safety factor for steel in fire conditions;

$k_{\mathrm{s}}(\theta)$ - a steel strength reduction factor for the given temperature θ for the required time of fire resistance;

M_{Ed} - the applied moment for ambient temperature design according to EN 1992-1-1;

$A_{\mathrm{s,prov}}$ - the cross-sectional area of tensile steel provided; and

$A_{\mathrm{s,req}}$ - the cross- sectional area of tensile steel required for the design at the ambient temperature according to EN 1992-1-1.

In this calculation, ratio $A_{\mathrm{s,prov}}/A_{\mathrm{s,req}}$ should not be greater than 1.3.

For determining the steel strength reduction factor $k_{\mathrm{s}}(\theta)$ at the temperature θ for the required fire resistance period, it is necessary to establish the temperature of the reinforcement θ first. Considering that the temperature of the steel reinforcement is assumed to be the same as the temperature of concrete in the observed fiber of the cross section, then the temperature of steel θ can be determined based upon temperature profiles for slabs given in Annex A, EN 1992-1-2: 2004, shown in the following *Figure*.

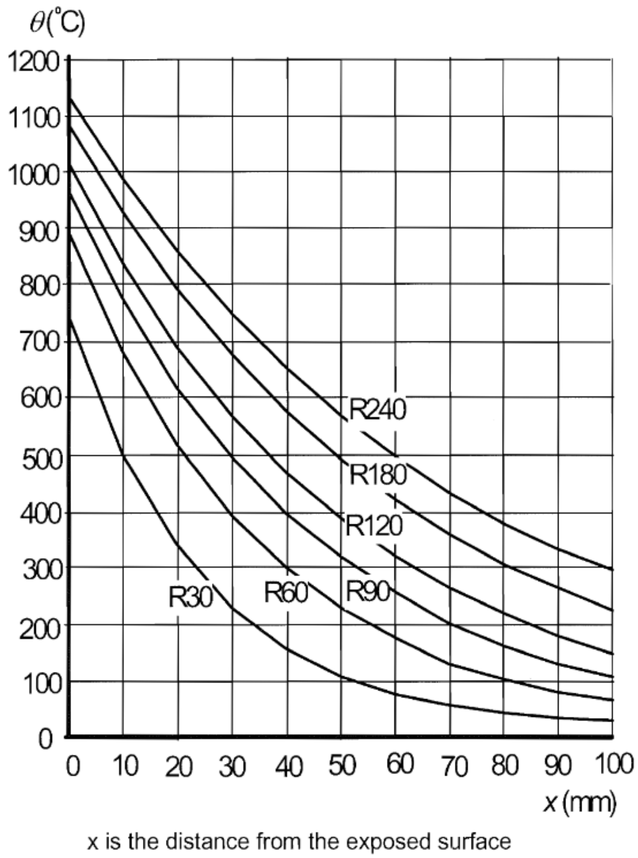


Figure 13 – Temperature Profiles for Slabs (height 200 mm) [28]

BRANZ TECHNICAL RECOMMENDATION No. 8 METHOD FOR FIRE ENGINEERING DESIGN OF STRUCTURAL CONCRETE BEAMS AND FLOOR SYSTEMS

BRANZ – The Building Research Association of New Zealand, Technical Recommendation No. 8 from 1991, provides a model for determining fire resistance of load-bearing beams and slabs made of reinforced or prestressed concrete, influenced by elevated temperatures. The basic design procedure was developed and verified through a large number of fire resistance tests of reinforced concrete elements sponsored by the Portland Cement Association - PCA. The significance of this method for determining fire resistance of concrete elements lies in its combination of the principles for determining the fire resistance given in ACI 216.1-97/ TMS 0216.1-97, but also applies the standard temperature-time curve according to ISO 834, for distinction from ACI 216.1-97/TMS 0216.1-97 which is based exclusively on the standard fire curve according to ASTM E 119.

This method can be applied for determining the fire resistance of concrete slabs, or when the following requirements are fulfilled:

- Slabs are made with normal weight concrete (density $\geq$ 20 kN/m^3) or lightweight concrete (density < 20 kN/m^3);
- Slabs are simply supported (with or without restraint against thermal expansion), or receive continuous support on both end bays and internal bays in framed structures;
- Slabs are reinforced by reinforcing bars, or prestressed tendons or strands;
- Fire resistance of slabs is necessary to be determined for 60, 90, 120, 180, or 240 minutes;
- Slabs are exposed to the ISO 834 fire curve heating regime.

Further considerations in this chapter will be limited only to reinforced concrete simply supported slabs (made of normal weight concrete) without restraint against the thermal expansion, which are the focus of this research.

Design procedure assumes that the following data are known:

- Depth of the slab, span and support conditions;
- Permanent and variable actions;
- Type of reinforcing steel and its cross-sectional position;

- Yield strength of reinforcing bars; and
- Concrete class, density, and 28-days cylinder compressive strength.

When considering behavior of reinforced concrete simply supported slabs without restraint against the thermal expansion in a fire, the load bearing capacity of the beam is reduced as a result of the reduction of mechanical properties of steel and concrete when exposed to high temperatures. Since actions are considered to be constant during the development of the fire, the applied bending moment remains the same during the entire fire development, and the bending failure occurs when the bending moment capacity of the slab becomes lower than the acting bending moment which is illustrated in *Figure 14.*

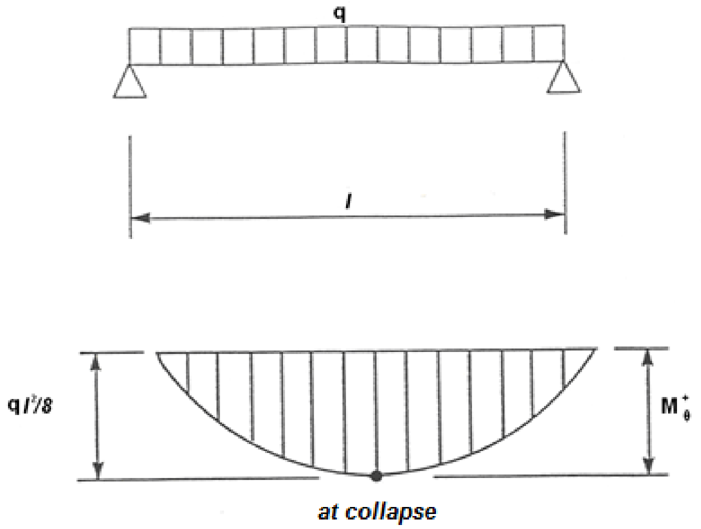


Figure 14 – RC Slab at Collapse [17]

The first step in determining the slab fire resistance is to deduce the effective concrete cover which is weighted average distance between the centers of individual steel bars and the nearest fire exposed surface of the slab (C_e).

It is feasible to determine the nominal temperature of the tensile steel (T_s) using the effective concrete cover from the diagram presented in *Figure 15* for the required period of fire resistance of the slab. This diagram was obtained in 1968 on the basis of series of fire tests carried out in the US (Portland Cement Association) by Abrams and Gustaferro.

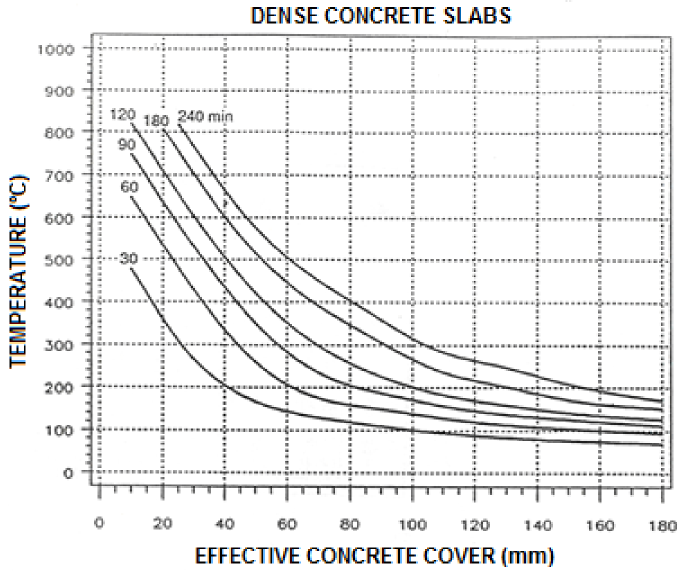


Figure 15 – Determination of the Temperature of Reinforcing Steel [17]

Based on the nominal temperature of the tensile steel (T_s), the yield strength of reinforcing steel at elevated temperatures is determined using the *Equation 22*, or proportion of the yield strength at elevated temperatures related to the yield limit at normal temperatures from *Figure 17*. Bilinear diagram "a" from *Figure 16* is used for reinforcing steel, while bilinear diagram "b" is used for prestressing steel.

$$\frac{f_y\,(T)}{f_y\,(20\,°C)} = \begin{cases} 1.0 & za\,T\ \leq 250\,°C \\ 1{,}53 - \dfrac{T}{470} & za\,T\ > 250\,°C \end{cases} \tag{22}$$

where

T - temperature in °C;

$f_y\,(20\,°C)$ - characteristic yield strength of reinforcing steel at ambient temperature;

$f_y\,(T)$ - yield strength of reinforcing steel at temperature T in °C.

This method considers the effective compression zone of concrete, where concrete has not reached a temperature greater than 750 ° C. Since all methods for determining the fire resistance of reinforced concrete elements assume that the temperature of concrete and the temperature of the reinforcement are identical at the same position, diagram in *Figure 16* can also be used to determine the temperature of concrete.

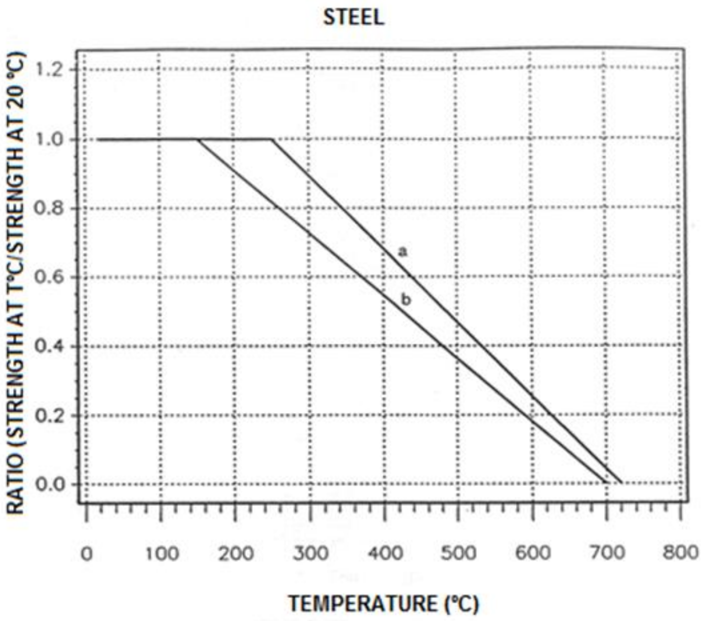


Figure 16 – Ratio of Steel Yield Strength at Elevated Temperatures [17]

In this case, the diagram shows the entrance of the vertical axis at 750 ºC, drawing a horizontal line which intersects the appropriate curve and then draws a vertical line which intersects the horizontal axis, giving the depth of surface layer which should be ignored [17]. However, since the compression zone of slabs is not exposed to the fire, the width of compression block is equal to 1.0 m for slabs. The depth of the equivalent rectangular stress block (a_Θ) is:

$$a_\theta = \frac{A_s \cdot f_{y\theta}}{0.85 \cdot f'_{c\theta} \cdot b_\theta} \tag{23}$$

where

a_θ - depth of the equivalent rectangular stress block at the required time of the fire resistance;

A_s - cross sectional area of tensile reinforcing steel provided;

$f_{y\theta}$ - yield stress of reinforcing steel at the temperature θ;

$f'_{c\theta}$ - reduced compression strength of concrete at the temperature θ;

b_θ - reduced width of concrete compression block at the temperature θ (it is equal to 1.0 m for slabs).

The normal weight concrete strength at elevated temperatures can be assessed using the following equation:

$$\frac{f_c'\ (T)}{f_c'\ (20\ °C)} = \begin{cases} 1{,}0 & za\ \ T \leq 350\ °C \\ 1{,}65 - 0.8\ \cdot \dfrac{T}{440} & za\ \ T > 350\ °C \end{cases} \tag{24}$$

where

T - temperature in °C;

$f'_c(20\ °C)$ - characteristic compression strength of concrete at the ambient temperature;

$f'_c(T)$ - compression strength of concrete at the temperature T.

For determining the concrete strength at elevated temperatures, the diagram in *Figure 17* can also be used, where bilinear diagram "a" is for normal weight concrete and bilinear diagram "b" is for lightweight concrete.

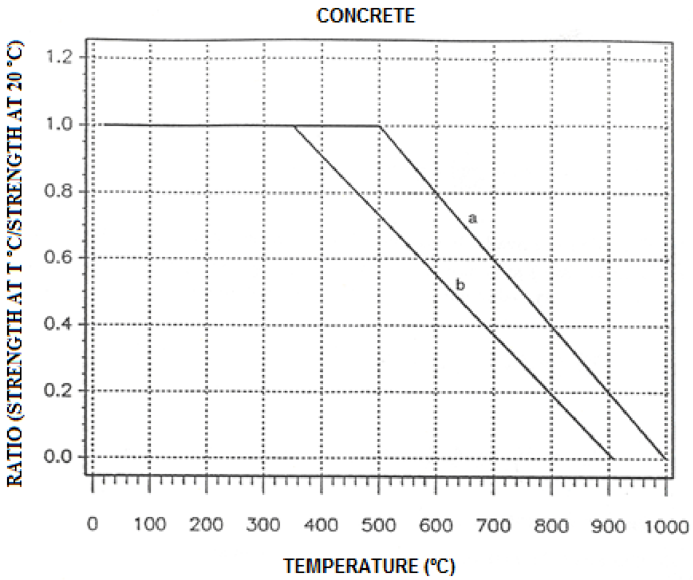


Figure 17 – Ratio of Concrete Compression Strength at Elevated Temperatures [17]

However, compression stress block of simply supported slabs exposed to the fire from the bottom in majority of cases has the temperature below 350 ºC, so the compression strength of concrete in case of fire is the same as characteristic concrete strength at the ambient temperature which simplifies the procedure for slabs.

Determination of the bending moment capacity M_θ^+ of the slab cross-section, given after the required time of exposure to the fire is carried out according to the following equation:

$$M_\theta^+ = A_s \cdot f_{y\theta} \cdot \left(d_\theta - \frac{a_\theta}{2}\right) \quad (25)$$

where

M_θ^+ - bending moment capacity of the slab cross-section;

A_s - cross sectional area of tensile reinforcing steel provided;

$f_{y\theta}$ - yield stress of reinforcing steel at the temperature θ;

d_θ - width of the compression stress block (1.0 m for slabs); and

a_θ - depth of the compression stress block.

According to this Technical Recommendation, but also many other regulations, the partial safety coefficient for permanent action in the fire environment is equal to 1.0. The safety factor for the variable action in case of fire for floors (domestic, office, parking and trafficable roofs) is 0.4; for floors in storages and other structures, it is 0.6, and 0 for non-trafficable roofs. Thus, design action in fire conditions is:

$$w = 1{,}0 \cdot g + \gamma_p \cdot p \quad (26)$$

where:

w - design action of the slab;

g - permanent action of the slab;

γ_p - safety factor for variable action in fire conditions;

p - variable action of the slab.

Then, the maximum applied bending moment to the simply supported slab with uniformly distributed load is:

$$M_a = \frac{w \cdot l^2}{8} \quad (27)$$

where

M_a - maximum applied bending moment to the simply supported slab with uniformly distributed load;

w - design action of the slab;

l - effective span of the slab.

If the bending moment capacity of the slab is greater than or equal to the maximum applied bending moment $M_{\theta}^{+} \geq M_a$, the slab has the required fire resistance; however, if the bending moment capacity of the slab is less than the maximum applied bending moment $M_{\theta}^{+} < M_a$, then the slab does not fulfill requirements for the required fire resistance period.

ACI/TMS 216.1 - CODE REQUIREMENTS FOR DETERMINING FIRE RESISTANCE OF CONCRETE AND MASONRY CONSTRUCTION ASSEMBLIES

The Standard Method for Determining Fire Resistance of Concrete and Masonry Structures ACI/TMS 216.1 reported by ACI/TMS Committee 216 gives a procedure for determining fire resistance of concrete elements based on the research conducted by Martin, Abrams, Gustaffero, Harmathy, Bletzacker, and other American researchers present in this field for many years.

This method for determining the fire resistance of concrete elements contains the necessary data for design of fire-resistant concrete structures, and for estimating the fire resistance of existing structures. The method also provides elements for determining the fire resistance of simply supported or continuous RC slabs. It also contains information on the behavior of concrete and concrete reinforcement at elevated temperatures, based on the exposure of elements to the standard fire curve ASTM E 119.

It is common for simply supported slabs to be reinforced by rebars or mashes at a specific distance from the bottom edge of the element. If the underside is exposed to the fire, as shown in *Figure 18*, the bottom side will experience greater elongation than the upper side, and this will result in a deflection.

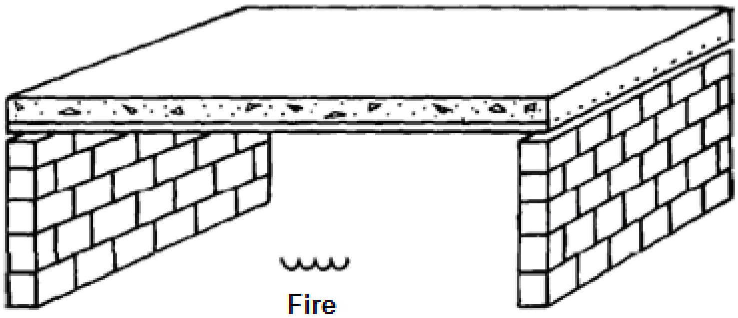


Figure 18 – RC Slab Exposed to Fire from the Underside [21]

With the increase of the temperature, the yield strength of steel and compression strength of concrete decreases. When the yield strength of the reinforcement is reduced due to the effects of high temperatures to the stress level in the reinforcement, it results in failure of the element due to bending.

Figure 19 shows the behavior of the simply supported slab exposed to the effect of fire from the underside of the slab.

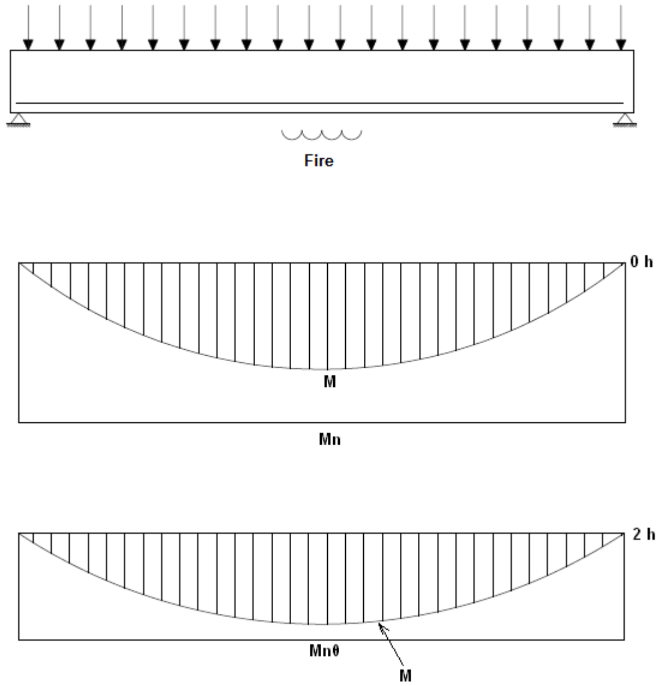


Figure 19 – Behavior of the Simply Supported Slab Exposed to the Effect of Fire from the Underside of the Slab [21]

If the reinforcement is straight and equal along the entire span of the slab, then the moment capacity is also equal along the entire span of the slab, and it is:

$$M_n = A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \tag{28}$$

where

A_s – cross sectional area of reinforcement provided;

f_y – characteristic yield strength of the reinforcing steel;

d - is the distance between the centroid of the reinforcing steel to the extreme compressive fiber;

a - is the depth of the equivalent rectangular compressive stress block at ultimate load.

The depth of the equivalent rectangular compressive stress block at the ultimate load can be determined as:

$$a = A_s \cdot f_y \cdot (0.85 \cdot f_c' \cdot b) \tag{29}$$

where

$f_c{}'$ - characteristic cylinder compressive strength of concrete;

b - 1.0 m for slabs.

If the slab is uniformly loaded, the moment diagram will be parabolic with a maximum value at midspan:

$$M = \frac{w \cdot l^2}{8} \tag{30}$$

where l is the span length and w is defined according to the Appendix C2.5 from ASCE 07, Minimum Design Loads for Buildings and Other Structures. This proposes load combinations for checking the capacity of a structure or structural element to withstand the effect of extraordinary events such as fires, which is characterized by low probability of occurrence and usually short duration. Load combinations include the following cases [4]:

$$1.2 \text{ Dead} + (0.5) \text{ Live or } 0.2 \text{ Snow} \tag{31}$$

$$(0.9 \text{ or } 1.2) \text{ Dead} + 0.2 \text{ Wind.} \tag{32}$$

It is generally assumed that during the fire, dead and live loads remain constant. However, the strength of materials is reduced so that the retained nominal moment strength is

$$M_{n\theta} = A_s \cdot f_{y\theta} \cdot \left(d - \frac{a_\theta}{2}\right) \tag{33}$$

in which θ signifies effects of elevated temperatures. Note that A_s and d are not affected, but $f_{y\theta}$ is reduced. Similarly, a_θ is reduced, but the concrete strength at the top of the slab f_c' is generally not significantly reduced.

It can be assumed that the flexural failure occurs when $M_{n\theta}$ decreases to M, and the conclusion is that the time of fire resistance depends on the load intensity and the behavior of the concrete reinforcement at high temperatures. In conclusion, the period of fire resistance of the observed slab depends on the time required to reach the critical temperature of the steel, which again depends on the applied protection of the reinforcement. The most commonly used protection is the concrete cover.

Fire resistance of simply supported slabs depends on the type of reinforcement used, the type of concrete depending on the aggregate, the intensity of the bending moment applied, and the distance of the reinforcement centroid from the fire exposed side of concrete denoted by "u". If the reinforcement is uniformly arranged along a tensile zone, the value "u" is determined as the average distance of individual bars from the fire exposed side of concrete.

Taking into account that the reinforcement index is

$$\omega = A_s \cdot f_y /(b \cdot d \cdot f_c') \tag{34}$$

and determining the ratio M/M_n, and also using "u", it is possible to determine the period of fire resistance of the slab from the diagram presented in *Figure 20*.

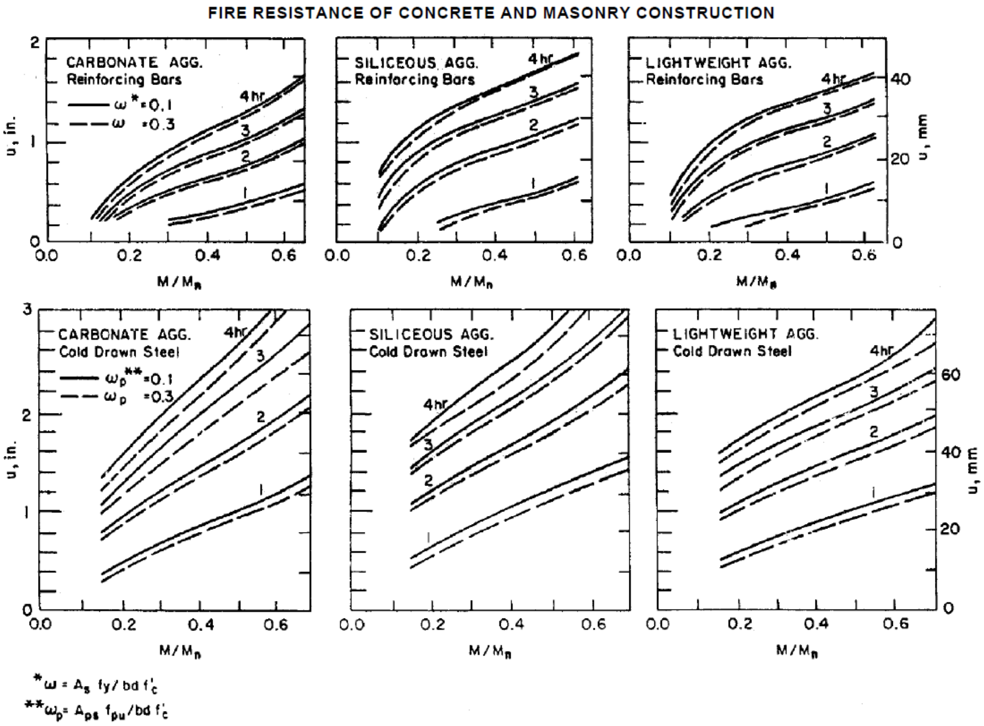


Figure 20 – Fire Resistance of Concrete Slabs as Influenced by Aggregate Type, Reinforcing Steel Type, Moment Intensity, and "u" [22]

FIRE RESISTANCE DETERMINATION OF RC SLABS

This study considers fire resistance determination of RC slabs using different methods: EN 1992-1-2:2004, Eurocode 2, Design of concrete structures, Part 1-2: Structural fire design - Tabulated data and Simplified calculation method for slabs, BRANZ Technical Recommendation No. 8 – Method for Fire Engineering Design of Structural Concrete Beams, and Floor Systems and ACI/TMS 216.1 – Code Requirements for Determining Fire Resistance of Concrete and Masonry Assemblies.

For the researching purposes, simply supported RC concrete slabs with spans of 3.0 m, 5.0 m, and 7.0 m were considered. Depths of slabs are 12, 15 and 17 cm respectively. All slabs were designed according to EN 1992-1-1, Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings. Actions taken into consideration are self-weight of slabs, flooring (1.5 kN/m^2), and variable load of 2 kN/m^2. Different concrete classes were used for each slab: C 20/25, C 30/37 and C 40/50. A concrete cover was also variated for each slab: 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0 cm. It should be noted that thicknesses of the concrete cover of 0.5 and 1.0 cm are not allowed for slabs according to Eurocode 2, part 1-1. However, given the construction situation in Bosnia and Herzegovina, it is not rare to encounter very thin concrete covers, or virtually no concrete covers at all in the actual construction practice, due to failure of the contractor involved, or poor inspection of the construction sites, so it is interesting to assess the fire resistance of such slabs.

All slabs were reinforced by welded ribbed meshes made of steel grade B500A, Ductility Class A, Yield = Re 500 MPa, or by straight ribbed bars made of steel grades B500A or St-500-b.

All slabs were exposed to the Standard Fire Curve ISO 834 for determining fire resistance according to EN 1992-1-2:2004, Eurocode 2, Design of concrete structures, Part 1-2: Structural fire design - Tabulated data and Simplified calculation method for slabs and BRANZ Technical Recommendation No. 8 – Method for Fire Engineering Design of Structural Concrete Beams and Floor Systems. To determine fire resistance according to the ACI/TMS 216.1 – Code Requirements for Determining Fire Resistance of Concrete and Masonry Assemblies, fire exposure is assumed according to ASTM E 119.

The results of fire resistance of RC slabs according to different methods are presented in *Tables 4-12.*

Table 4 – Fire Resistance of RC Slab, Span 3 m, Depth 12 cm, C20/25

FIRE RESISTANCE OF SLAB **depth 12 cm, span 3 m, C 20/25** **$q_d = 9.075$ kN/m^2**								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed, fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	0.825	7.14	R 221	6.5	$R_\emptyset$	$R_\emptyset$	0 min	< 60 min
1.0	1.275		R 238	5.5	R 30	R 30	30 min	< 60 min
1.5	1.850		R 257	7.0	R 30	R 30	30 min	< 60 min
2.0	2.350		R 257	7.0	R 60	R 60	60 min	< 60 min
2.5	2.800		R 283	6.0	R 60	R 60	60 min	60 min
3.0	3.325		R332	6.5	R 90	R 90	90 min	60 min

Table 5 – Fire Resistance of RC Slab, Span 3 m, Depth 12 cm, C30/37

FIRE RESISTANCE OF SLAB depth 12 cm, span 3 m, C 30/37 $q_d = 9.075$ kN/m^2								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed, fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	0.825	7.14	R 221	6.5	$R_{\emptyset}$	$R_{\emptyset}$	0 min	< 60 min
1.0	1.300		R 238	5.5	R 30	R 30	30 min	< 60 min
1.5	1.850		R 257	7.0	R 30	R 30	30 min	< 60 min
2.0	2.350		R 257	7.0	R 60	R 60	60 min	< 60 min
2.5	2.800		R 283	6.0	R 90	R 60	60 min	60 min
3.0	3.325		R 332	6.5	R 90	R 90	90 min	60 min

Table 6 – Fire Resistance of RC Slab, Span 3 m, Depth 12 cm, C40/50

FIRE RESISTANCE OF SLAB **depth 12 cm, span 3 m, C 40/50** **q_d = 9.075 kN/m^2**								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed, fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	0.825	7.14	R 221	6.5	$R_Ø$	$R_Ø$	0 min	< 60 min
1.0	1.275		R 238	5.5	R 30	R 30	30 min	< 60 min
1.5	1.775		R 228	5.5	R 60	R 30	30 min	< 60 min
2.0	2.350		R 257	7.0	R 60	R 60	60 min	< 60 min
2.5	2.800		R 283	6.0	R 90	R 60	60 min	60 min
3.0	3.300		R 283	6.0	R 90	R 90	60 min	60 min

Table 7 – Fire Resistance of RC slab, Span 5 m, Depth 15 cm, C20/25

FIRE RESISTANCE OF SLAB depth 15 cm, span 5 m, C 20/25 $q_d = 10.08$ kN/m^2								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed,fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	0.95	22.03	R 636	9	R 30	R$_\varnothing$	0 min	< 60 min
1.0	1.45		R 636	9	R 30	R 30	30 min	< 60 min
1.5	1.95		R 636	9	R 60	R 30	30 min	< 60 min
2.0	2.45		R 636	9	R 60	R 60	60 min	< 60 min
2.5	3.00		R 785	10	R 60	R 90	60 min	60 min
3.0	3.50		R 785	10	R 90	R 90	90 min	60 min

Table 8 – Fire Resistance of RC Slab, Span 5 m, Depth 15 cm, C30/37

FIRE RESISTANCE OF SLAB **depth 15 cm, span 5 m, C 30/37** **q_d = 10.08 kN/m^2**								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed, fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	0.95	22.03	R 636	9	R 30	$R_{\emptyset}$	30 min	< 60 min
1.0	1.45		R 636	9	R 30	R 30	30 min	< 60 min
1.5	1.95		R 636	9	R 60	R 60	30 min	< 60 min
2.0	2.50		R 785	10	R 90	R 90	90 min	60 min
2.5	3.00		R 785	10	R 90	R 90	90 min	60 min
3.0	3.50		R 785	10	R 120	R 90	90 min	60 min

Table 9 – Fire Resistance of RC slab, Span 5 m, Depth 15 cm, C40/50

FIRE RESISTANCE OF SLAB depth 15 cm, span 5 m, C 40/50 $q_d = 10.08$ kN/m^2								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed, fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	0.95	22.03	R 636	9	R 30	$R_{\varnothing}$	30 min	< 60 min
1.0	1.45		R 636	9	R 30	R 30	30 min	< 60 min
1.5	1.95		R 636	9	R 60	R 30	30 min	< 60 min
2.0	2.50		R 636	9	R 60	R 60	60 min	< 60 min
2.5	3.00		R 785	10	R 90	R 90	60 min	60 min
3.0	3.50		R 785	10	R 120	R 90	90 min	60 min

Table 10 – Fire Resistance of RC Slab, Span 7 m, Depth 17 cm, C20/25

FIRE RESISTANCE OF SLAB depth 17 cm, span 7 m, C 20/25 q_d = 10.76 kN/m^2								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed, fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	1.10	46.12	R 1130	12	R 30	R 30	30 min	< 60 min
1.0	1.60		R 1130	12	R 30	R 30	30 min	< 60 min
1.5	2.20		Ø 14/12.5 cm	14	R 60	R 60	60 min	< 60 min
2.0	2.70		Ø 14/12.5 cm	14	R 90	R 60	60 min	< 60 min
2.5	3.20		Ø 14/10 cm	14	R 120	R 90	90 min	60 min
3.0	3.70		Ø 14/10 cm	14	R 120	R 90	90 min	60 min

Table 11 – Fire Resistance of RC Slab, Span 7 m, Depth 17 cm, C30/37

FIRE RESISTANCE OF SLAB **depth 17 cm, span 7 m, C 30/37** **q_d = 10.76 kN/m^2**								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed, fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	1.10	46.12	R 1130	12	R 30	R 30	30 min	< 60 min
1.0	1.60		R 1130	12	R 30	R 30	30 min	< 60 min
1.5	2.20		Ø 14/12.5 cm	14	R 60	R 60	30 min	< 60 min
2.0	2.70		Ø 14/12.5 cm	14	R 90	R 60	60 min	60 min
2.5	3.20		Ø 14/12.5 cm	14	R 90	R 90	60 min	60 min
3.0	3.70		Ø 14/10 cm	14	R 120	R 90	90 min	60 min

Table 12 – Fire Resistance of RC Slab, Span 7 m, Depth 17 cm, C40/45

FIRE RESISTANCE OF SLAB depth 17 cm, span 7 m, C 40/50 $q_d = 10.76$ kN/m^2								
Concrete cover (cm)	**a_s (cm)**	**$M_{Ed, fi}$ (kNm/m')**	**Reinforcement**	**Bar diameter (mm)**	**EN 1992-1-2 Simplified Method for Slabs**	**EN 1992-1-2 Tabulated data**	**BRANZ TR 8**	**ACI/TMS 216.1**
0.5	1.10	46.12	R 1130	12	R 30	R 30	30 min	< 60 min
1.0	1.60		R 1130	12	R 30	R 30	30 min	< 60 min
1.5	2.10		R 1130	12	R 60	R 60	30 min	< 60 min
2.0	2.60		R 1130	12	R 60	R 60	60 min	< 60 min
2.5	3.20		Ø 14/12.5 cm	14	R 90	R 90	60 min	60 min
3.0	3.70		Ø 14/10 cm	14	R 120	R 120	90 min	60 min

CONCLUSION

This parallel comparison research for determining fire resistance of reinforced concrete slabs of different spans and depths, made of three different concrete classes, with variations of concrete covers ranging from 0.5 m to 3.0 cm, using different methods, provided the following results:

- Fire resistance periods of RC slabs considered and determined by four different methods were similar, but not the same. However, maximum difference in fire resistance periods was up to 60 minutes;
- Both methods of EN 1992-1-2 (Tabulated Data and Simplified Method for Slabs) provided equal or similar fire resistance periods, although Tabulated Data provided shorter fire resistance periods in some cases, which was expected if take into consideration that Simplified Method of calculation provided an extension to the use of the tabular method, as defined in EN 1992-1-2. Maximum difference was 30 minutes;
- In majority of cases, BRANZ TR 8 had the same fire resistance periods as determined by EN 1992-1-2 Tabulated Data Method. The difference was not more than 30 minutes, but was more conservative when using BRANZ TR8;
- Standard Method for Determining Fire Resistance to Concrete and Masonry Structures ACI/TMS 216.1, using diagrams from *Figure 20* was not sufficiently sensitive for determining fire resistance periods of slabs as other methods were. Differences were up to 60 minutes, and there was not a clear determination of fire resistance period below 60 minutes. It is also necessary to highlight that results obtained using ACI/TMS 216.1 were not fully comparable with other three methods, since the fire exposure of Standard Fire (ASTM E 119) was not the same as Standard Fire ISO 834-1, although without significant differences in temperatures. The reason this method was considered in this research was its wider and holistic meaning of the term “fire resistance”, where fire resistance and structures did not recognize borders, countries, or different standards;
- The research also confirmed the fact that concrete class had minor influence on the fire resistance period of slabs, while thickness of concrete cover significantly affected the period of fire resistance of slabs;
- Maximum fire resistance period of slabs observed in this research was 120 minutes.

REFERENCES

[1] American Concrete Institute, 1989, ACI 216R-89," Guide for Determining the Fire Resistance of Concrete Elements, Farmington Hills, Michigan, USA;

[2] Ali E. Y, 2010, "Limit-Analysis of R/C Slabs Exposed to Standard Fires: Application of the Yield-Line Method", Master Thesis, Structural Engineering Department, Politecnico Di Milano, Italy;

[3] Cement Concrete & Aggregates Australia, 2010 "Fire Safety of Concrete Buildings", Level 6, 504 Pacific Highway St Leonards NSW Australia 2065, Sydney, Australia , ISBN 978-1-877023-27-9;

[4] Concrete Reinforcing Steel Institute CRSI, 2015, "Fire resistance of RC Buildings", CRSI Technical Note ETN-B-1-16, Schaumburg, Illinois, USA;

[5] Džidić S, 2014, "Određivanje otpornosti na požar AB ploča izloženih parametarskim krivim požara", V International Conference, "Civil Engineering – Science and Practice - GNP 2014", February 2014, Žabljak, Montenegro, Book of Proceedings, ISBN 978-86-82707-23-3 COBISS.CG-ID 24170256;

[6] Džidić S, 2015. "Otpornost betonskih konstrukcija na požar", International BURCH University Sarajevo, October 2015, Sarajevo, Bosnia and Herzegovina, ISBN 978-9958-834-47-9; COBISS.BH-ID 22444550;

[7] Fraser A. S, Jones A. E. K, 2009, "Structural Fire Design", The Concrete Centre, Riverside House 4, Medows Business Park, Station Approach, Blackwater, Camberly Surren, GU 17 19AB;

[8] Kodur V, 2014, "Properties of Concrete at Elevated Temperatures", Hindawi Publishing Corporation, ISRN Civil Engineering, Article ID 468510, Volume 2014, March 2014;

[9] Lawson J. R, Phan L. T, Davis F. L, 2006, "Mechanical Properties of High Performance Concrete after Exposure to Elevated Temperatures", National Institute of Standards and Technology, NIST, Gaithersburg, Maryland, 2000;

[10] Levesque A, 2006, Fire Performance of Reinforced Concrete Slabs, Master Thesis, Worchester Polytechnic Institute, USA;

[11] Lehner S, 2005, "European Fire Classification of Construction Products, New Test Method "SBI", and Introduction of the European Classification System

into German Building Regulations", Otto-Graf-Journal Vol. 16, 2005, 151-165;

[12] Lie, T. T. (Eds.), 1992, "Structural Fire Protection, (Technical Report No. 78)", New York: American Society of Civil Engineers, USA;

[13] Mešić H, Džidić S, 2015. "Uticaj debljine pokrovnog sloja betona na otpornost na požar armiranobetonskih ploča prema Eurocode 2", 10th International Conference "RIM 2015", Development and Modernization of Production, October 4-6, 2015, Dubrovnik, Croatia, Book of Proceedings ISBN 978-9958-624-41-4, COBISS.BH.ID 22420486;

[14] Purkiss J. A, 2007, "Fire Safety Engineering Design of Structures", Second Edition, Butterworth-Heinemann is an imprint of Elsevier, Linacre House, Jordan Hill, Oxford OX2 8DP, UK, Burlington, MA 01803, USA;

[15] Schleich J, 2005, "Fire Actions in Buildings" Leonardo Da Vinci Project CZ/02/B/F/PP-134007, Development of Skills Facilitating Implementation of Eurocodes, Handbook 5, Design of Buildings for the Fire Situation, Luxembourg;

[16] Wade C, 1991, "Fire Engineering Design of Reinforced and Prestressed Concrete Elements (Study Report No. 33)", Judgeford: BRANZ, The Resource Centre for Building Excellence;

[17] Wade C, 1991, "Method for Fire Engineering Design of Structural Concrete Beams and Floor Systems (Technical Recommendation No. 8)", Judgeford: BRANZ, The Resource Centre for Building Excellence;

[18] Wade C, 1992, "Fire Resistance of New Zealand Concretes (Study Report No. 40)", Judgeford: BRANZ, The Resource Centre for Building Excellence;

[19] Wade C.A, 1994, "Performance of Concrete Floors Exposed to Real Fires", Journal of Fire Protection Engineering, New Zealand;

[20] Wong Y. L , 2013, "Fire Resistance Design and Protection for Concrete", Standing Committee on Concrete Technology Annual Concrete Seminar 2013, Hong Kong, April 2013.

STANDARDS

[21] American Concrete Institute, 1989, ACI 216R-89," Guide for Determining the Fire Resistance of Concrete Elements, Farmington Hills, Michigan, USA;

[22] American Concrete Institute, 1997, ACI 216.1-97/TMS 0216.1-97, Standard Method for Determining Fire Resistance of Concrete and Masonry Construction Assemblies, Farmington Hills, Michigan, USA;

[23] American Concrete Institute, 2014, ACI /TMS 216.1-14, Standard Method for Determining Fire Resistance of Concrete and Masonry Construction Assemblies, Farmington Hills, Michigan, USA;

[24] ASTM International – ASTM, 2014, Standard Test Methods for Fire Tests of Building Construction and Materials (ASTM E119-14); ASTM International, Conshohocken, Pennsylvania, USA;

[25] BAS EN 1991-1-1:2015 - Eurocode 1 - Actions on structures: Part 1-1: General actions - Densities, self-weight and imposed loads for buildings, Institute for Standardization of Bosnia and Herzegovina, Istočno Sarajevo, Bosnia and Herzegovina;

[26] BAS EN 1991-1-1/NA:2016 - Eurocode 1 - Actions on structures: Part 1-1: General actions - Densities, self-weight and imposed loads for buildings - National annex, Institute for Standardization of Bosnia and Herzegovina, Istočno Sarajevo, Bosnia and Herzegovina;

[27] BAS EN 1992-1-1:2017 Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings, Institute for Standardization of Bosnia and Herzegovina, Istočno Sarajevo, Bosnia and Herzegovina;

[28] BAS EN 1992-1-2:2017- Eurocode 2 - Design of concrete structures: Part 1-2: General rules - Structural fire design, Institute for Standardization of Bosnia and Herzegovina, Istočno Sarajevo, Bosnia and Herzegovina;

[29] BAS ISO 834-1:2007 , Fire-resistance tests - Elements of building construction – Part 1: General requirements Institute for Standardization of Bosnia and Herzegovina, Istočno Sarajevo, Bosnia and Herzegovina.

STUDY 2

HIGH-STRENGTH CONCRETE (HSC) AND POSSIBILITIES FOR PRODUCTION IN BOSNIA AND HERZEGOVINA

Ilda Kovačević

Sanin Džidić

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

It is hard to define high-strength concrete (HSC) with one unique number, or create any strict border between conventional normal strength concrete and high–strength concrete. As long as achieved concrete or target strength is about the same quality as the local material, curing conditions, size and age of testing specimens impose the fact that nor unique nor unified definition of high–strength concrete is neither possible nor necessary. Another factor in defining ranging lines of high–strength concrete is also a demand for specific strengths or performances of concrete. In the specific case of the USA or some rapidly growing Asian country or city, 95 MPa high–strength concrete is available in most of concrete plants, and at same time it is economically and cost efficient. [12]

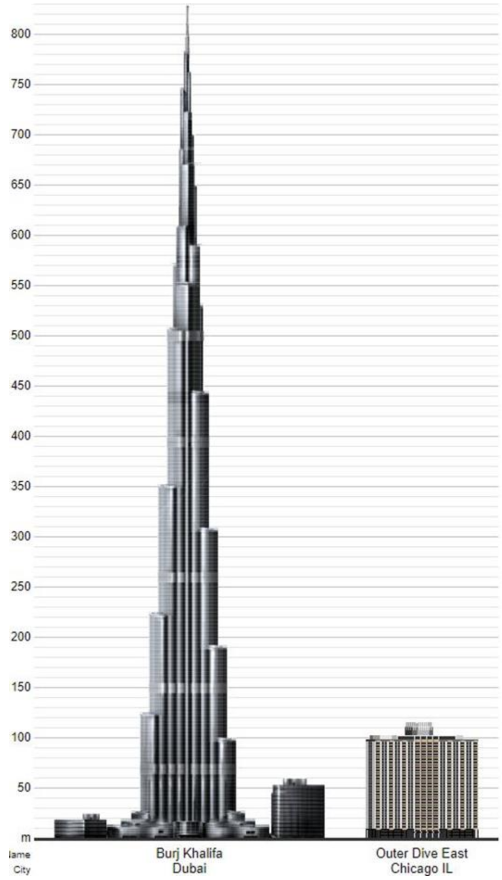


Figure 1- The Last Completed Super High–Rise, Burj Khalifa, 2010 (left) and the First Completed High- Strength Concrete High–Rise, Outer Drive East 1963 [38]

On the other hand, situation in Balkan area is totally opposite. Abilities to use high–strength concrete in this area is not even adequately researched, and the top limit of concretes’ strength may appears to be up to 50 MPa, which corresponds to weak economical and cost efficiency. However, in different standards there are some differences in classifications of concrete up to the characteristic compression strengths. By BAS EN 206:2014 [33], normal weight and heavy weight concretes are divided into sixteen classes according to their compressive strengths; high–strength concrete is in range between C55/67 and C100/115.

Terms high strength concrete and high–performance concrete were commonly used as synonyms, which was acceptable at the early beginnings. However, in the contemporary concrete technology, this interchangeable use of these terms is not acceptable.

High–strength concrete commonly refers to the increase in compressive strength of concrete mixtures, while high–performance concrete refers to the increase of all concrete’s properties, with accent on mechanical properties, durability, workability, permeability etc. which is more than just increase of strength. [12]

HISTORY OF HIGH–STRENGTH CONCRETE (HSC)

Commonly, the periphrastic high–strength concrete is introduced as new material or as a result of new technological development. Although such periphrasis may be taken as correct, term high–strength concrete and practice of creating high–strength concrete occurred many decades ago. Dating back to 1950s, concrete with compressive strength of 34 – 35 MPa, was considered to be high–strength concrete. However, when compared to contemporary daily routine in concrete solutions, designed compressive strength of 34–35 MPa, at 28th day of curing is one of the most common examples of conventional or so called normal strength concrete.

More specific and more scientific approach to the subject of high–strength concrete occurred in the 1960s. Newly developed high–strength concrete with compressive strength of 41 to 52 MPa, rapidly spread through the construction sites across the USA. For high–strength concrete technology, sixties of the last century were crucial turning point because all experimental studies of technological development were aiming for the achievements of the desired results.

In the early sixties, Japan was a place where the first superplasticizers were developed. Formaldehyde condensates of beta naphthalene sulfonates, were developed by Dr. Hattori. These superplasticizers had primary function to reduce water demand in production of high–strength concrete. Product created was named Mighty 150, which could decrease water usage up to 30 per cent. Along with superplasticizers, use of another supplementary material for high–strength concreting developed was silica fume or so called microsilica; micro filler in between cement particles, a by-product of ferro–alloy industry.

Although invention of superplasticizers and silica fume took place in Japan and Germany, most of the credits in HSC development for wide use went to Chicago, United States. During the early sixties, Chicago was a place which accelerated development of high–strength concrete and increased that day available concrete's compressive strength of 35 MPa to 41 MPa for 40–storeys high–rise buildings. An engineering step forward pioneered the use of high–strength concrete in Chicago on the Outer Drive East high–rise building. [12, 36]

In the seventies of the last century, use of high–strength concrete gained wider and more diverse application. Greater demand for more resistant and more durable material forced the development which finally resulted in greater achievable concrete strength and overall performance of concrete.

By 1971, there was a high demand for a structural material which could resist very aggressive environment conditions in the deepest zone of the North Sea in Norway.

Thus, concrete with characteristic compressive strength of 80 MPa, found its appliance in oil platforms, across Ekofisk oil fields in the North Sea. Although technology produced high strength concrete of 80 MPa, it was still not economical for usage and wide application in construction.

Figure 2 - High–Strength Concrete Oil Platform, in Ekofisk Oil Fields, Norway [51]

Although use of high–strength concrete of the concrete class of 80 MPa, was uneconomical for uses in general construction, higher strength improved resistance to these extremely aggressive environments, and higher efficiency for large spans and greater overall performance put high–strength concrete on the first place for construction of bridges, river dams, marina pier and terminals all around the world.

Figure 3 - Prestressed High–Strength Concrete Prefabricated Truss for Bridge Iwahana, Japan [48]

Among European countries leaders in usage of high–strength concrete for bridge structures are Norway and France. In Norway, back in 1988/89 bridge SandhornØya was the first example of bridge structured with heavy weight high–strength concrete. Total length of the bridge is 374 meters, with three spans of 110 – 154 – 110 meters. Another such example is located in France, bridge Ile de Re, 3 kilometres long and connected with spans of 110 meters. Construction lasted for two years, and prefabricated high–strength concrete elements with compressive strength of 60 MPa were used.

Figure 4 - Ile de Re Bridge, Prefabricated High–Strength Concrete Structure [49]

The USA, also constructed numerous bridges, river dams, marina piers and terminals; however their main focus was on structuring of high–rise buildings, multi storey garages, shopping malls etc. For instance, it was almost mandatory for high–rise buildings in Chicago to be structured with high–strength concrete. In 1972, from previous 41 MPa, concrete's strength already increased to 52 MPa for structuring of 52–storey Mid–Continental Plaza. It is important to mention that production and application of high–strength concrete used to structure Mid-Continental Plaza, was more of an economical choice rather than a solutions. Achievable strength of concrete and all performances of concrete were increasing year after year with correspondence to cost efficiency. Such development of chemical admixtures and other supplementary materials culminated in 74–story high Water Tower Palace, in 1976. Water Tower palace, was the world's highest high–rise structure in that period, designed as concrete structure reaching compressive strength of 62 MPa.

After all, it is more than obvious that American Concrete Institute can take all credits for the rapid development of high–strength concrete and actual exposing of high–strength concrete to a wider market for application in most of the high-rise buildings worldwide.

Nowadays, high–strength concrete is in wide use all around the developed world, and it is more than common to find concrete plants which can catch up with the production of concrete with compressive strength of 95 MPa, on daily basis. [12]

ADVANTAGES AND DISADVANTAGES OF HIGH–STRENGTH CONCRETE (HSC)

High–strength concrete was developed as better and as structural material of higher quality when compared to normal strength concrete. Therefore it has many benefits, both in performance and cost efficiency, so HSC advantages are as follows:

- Reduction in structural element size;
- Reduction in amount of longitudinal reinforcement and compression members, focusing on slenderer columns;
- Higher strength and better performance leads to larger spans and decrease of total number of beams, columns etc.;
- Decreased time necessary for concrete's formwork due to early strength development;
- Decrease in concrete cover due to lower permeability;
- Long performance under the most critical action combinations;
- Lower creep and shrinkage with higher resistance for freezing and thawing;
- Increased resistance to very aggressive environments;
- Decreased axial shortening, buckling of supporting elements;
- Increased rentable space, due to slenderer and thinner elements, but also decreased number of supporting elements due to larger spans;
- Decreased permanent action of self–weight of structure;
- Decreased maintenance and repair costs;
- Greater stiffness due to higher modulus of elasticity with high compressive and flexural strengths.

Although high–strength concrete has many advantages as a material, it also has disadvantages which may occur due to some impurities or even as a consequence of some advantages mentioned above. High strength concrete disadvantages are:

- Bond strength between cement paste and aggregate does not increases with the same acceleration as compressive strength;
- High–vibration are required for better compaction, and to exclude possible segregations;

- Minimal concrete cover for reinforcement protection may prevent the use of maximum benefits in reduction of element sizes;
- Available prestressing may be inadequate for the maximum use of high–strength concrete's strength;
- High–strength concrete requires very detailed, precise and careful material selection and does not accept any impurities;
- Due to low W/C ratio, high–strength concrete requires special curing and installation or placement;
- There is a possibility of decrease in stiffness, whereas modulus of elasticity does not respectively increase with concrete's strength, therefore use of high–strength concrete may provide slenderer elements but with lower stiffness which may lead to stability problems, whereas solution lays in very precise choice of structural systems. [12]

CONSTITUENT MATERIALS FOR HIGH–STRENGTH CONCRETE (HSC)

Like a conventional normal–strength concrete, HSC also contains constituents, or in other words raw materials. Materials which participate in high–strength concrete proportioning are: cement, supplementary cementitious materials, fly ash, silica fume and/or some other mineral admixtures, aggregates of the best quality and of high compressive strengths; including dolomites, granites, quartz etc., as well as superplasticizers or some other types of chemical admixtures.

It is important for high–strength concrete to have raw materials of the highest quality without any compromises for marginal or lower qualities. If raw high quality materials are well proportioned and combined, it is possible to produce high-strength concrete with long lasting compressive strength and other mechanical properties. [12]

Cement is one of the inevitable materials for concrete mixtures, and plays one of the most important roles in concrete proportioning and production. The most common type is Portland cement, which, however, has different variations with different qualities and properties.

As previously mentioned, for high– strength concrete all constituent materials must be of high quality, without any impurities or compromises. Therefore, choice of cement requires complex studies and considerations of the behaviour of cement, its strength and all other properties in order to use the highest performances available from the cement.

Portland cement is material produced in specialized manufactories with different conditions and processes of production, all of which have effect on properties of cement and its qualities. Cement properties depend on burning temperatures, cooling duration and oxygen availability. These conditions also influence possible impurities which affect strength and performances. In Europe (EN) the most “common cement types” are classified as follows:

- CEM I Portland cement – comprising Portland cement and up to cement 5% of minor additional constituents;
- CEM II Portland – Portland - composite cement – Portland cement and up to composite cement 35% of other single constituents;
- CEM III Blast furnace cement – Portland cement and higher cement percentages of blast furnace slag;

- CEM IV Pozzolanic cement – Portland cement and up to 55% of cement pozzolanic constituents;
- CEM V Composite cement - Portland cement, blast furnace slag cement and pozzolana or fly ash. [4]

Figure 5 - Cement Types Produced by Kakanj Cement (Heidelberg Cement Group) [45]

Generally, all of these types of Portland cement proved to be suitable in production of concrete of compressive strength up to 60 MPa at the 28th day of age. However, to achieve higher strength with respective increase in performance and workability it is necessary to design and study reactions between additional chemical and mineral admixture.

Along with Portland cement, use of Blended hydraulic cement in production of high–strength concrete is common. Blended hydraulic cement is mixture of Portland cement and other supplementary cementitious materials, also named mineral admixtures. Benefits of Blended hydraulic cements lay in lower rate of heat development, higher strength, lower permeability, increased durability and overall performances. [12]

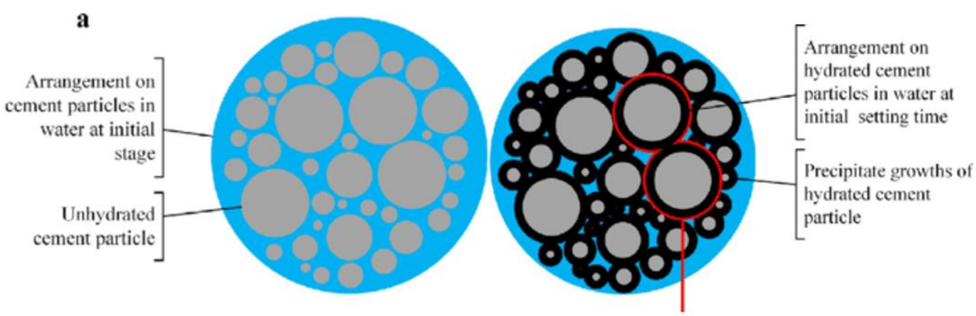


Figure 6 - Schematic View, of Cement Particles in Water at the Initial Stage (left), and at the Initial Setting Time (right) [46]

Credits for accelerating the development of high strength concrete technology go to the mineral admixtures, usually denoted as supplementary cementitious materials (SCM). These are the materials which developed and increased concretes' performance and strength of both fresh and hardened concrete. Generally, mineral admixtures are siliceous and alumina siliceous materials which with the addition of water chemically react with calcium hydroxide in order to perform cementitious properties. The most common types used in preparation of high – strength concrete are fly ash, cement slag and silica fume, while less in use are ultra-fly – ash, volcanic ashes, met - kaolin, diatomaceous earths and calcined natural pozzolans. Benefits of blended hydraulic cement in lower permeability, higher strength, and lower heat of hydration are also benefits of mineral admixture (SCM). [12]

Figure 7 - Common Mineral Admixtures – Supplementary Cementitious Materials for High–Strength Concrete [41]

Fly ash is commonly added to all concretes for higher performances. When combine fly ash and slag cement with Portland cement, it may create concretes with compressive strengths of 70 MPa. [12] However, it is not rare to see fly ash and slag cement replace Portland cement, or more than 50 percent of cements' content; in that specific case, fly ash and cement slag are considered as cementitious materials and not a mineral admixtures. On the other hand, silica fume, super fly–ash are defined by the specific function of admixture. These types of mineral admixtures never replace Portland cement, but strictly focus on increasing concrete's properties. [12]

Supplementary cementitious materials improve both fresh and hardened concrete, and are crucial for development and production of high–strength concrete. Even tough supplementary cementitious materials bring benefits, they all have to be evaluated in order to prevent some negative chemical reactions.

Fly ash is the most common type of SCM and by–product of combustion of pulverized coal; it is spherically shaped and glassy residue. Low, intermediate and high–fly ashes are usually used, able to correspond to the amount of calcium oxide in total mass. For instance if fly ash has less than 10 per cent of calcium oxide, such fly ash is considered as low–fly ash; if the calcium oxide is between 10 to 20 per cent, it is considered an about intermediate fly–ash while high–fly ash has more than 20 per cent of the calcium oxide in total mass. Primary difference is that low calcium oxide fly ash does not have hydraulic (cementing) properties, and therefore as the content of calcium oxide increases hydraulic properties increase as well. In order to decide which type of fly ash is required and amount of fly ash participating in concrete proportioning, experimental studies are required.

However, some common practices showed that optimal quantity of fly ash in the mix design of conventional concrete in order to achieve maximum compressive strength at the 28th day of age is approximately 25 per cent of the mass of total cementitious material content. However, if consider high–strength concrete, such percentages increase up to 40 – 50 per cent of total cementitious materials.

Experimental studies about how to enhance fly ash’s advantages in concrete mixture and production were conducted, and they resulted in successful production of high–strength concrete with compressive strength of 80 MPa, without use of silica fume, but rather of 30 – 40 per cent of fly ash of total cementitious materials mass.

Therefore fly ash is definitely one of the most important products of concrete technology, which enhances and increases concrete’s workability, while reducing water content.

Silica fume or micro silica is a by–product of silicon metals and ferrosilicon alloys, generated during reduction of quartz in the production of silicon metals and ferrosilicon alloys. This ultra–fine non–crystalline by–product, enabled widespread of high–strength concrete and the ability to produce ultra–high–strength concrete at all. [12]

Being a by–product of any manufacturing process indicates that characteristics of the specific by–product may vary from manufacturer to manufacturer. Therefore, chemical and physical characteristics of silica fume vary due a type of raw material used during manufacturing process and also the production technology. Generally, silica is described as grey to black dust. Silica fume grains are approximately 100 times smaller than Portland cement grains with sizes of 0.1 to 0.3 μm.

Although silica fume or micro silica has numerous advantages, its fineness may require higher percentage of water which may cause a decrease in workability and other desired properties if high–range water reduction admixtures are not added.

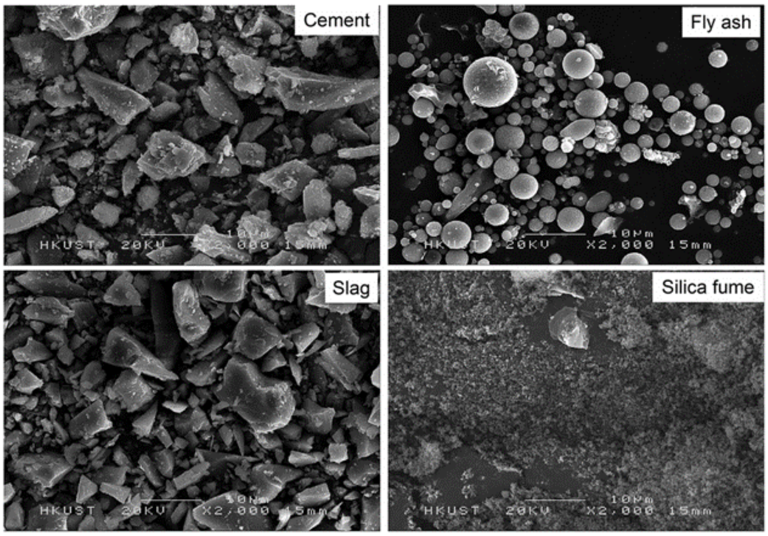


Figure 8 - Microscopic View of Cement, Fly Ash, Slag and Silica Fume Particles at the Same Scale – Enhancing Fineness of the Silica Fume Particles and Its Role in Micro Filling [42]

Silica fume as composite material of concrete mixture affects both fresh and hardened concrete and is also effective at concrete's early and late ages. High–strength concrete generally uses around 5 – 10 percent of total cementitious materials, where such proportions proved to be extremely effective. As a materials which gained large attention in concrete technology, silica fume has numerous advantages. Small particles of silica fume lubricate concrete and respectively increases its pump–ability, and enables easier concrete's placement around reinforcing bars, meshes, sections etc.

Use of silica fume decreases permeability, which means that silica fume fills in pores ad voids with in concrete. This way silica fume increases concrete's duration and increases its resistance to aggressive environmental conditions.

The principle of micro–filling with silica fume benefited in strengthening the bond between coarse aggregate and cement paste, with the ability of achieving compressive strength of over 105 MPa. Silica fume also tends to be efficient in reduced demand of other cementitious materials, for instance 1 kg of silica fume may replace 2 to 5 kg of cement, while the remaining content of water. [12]

Advantages of silica fume in concrete's later years primarily rely on increased resistance to environmental conditions and a decrease of permeability. Desired higher strengths are generally developed at early ages. Therefore, general agreement between researches is that silica fume's influence on concrete's strength develops during early ages. For instance, high–strength concrete without silica fume after 7 days of moist curing would have 34 – 57% lower strength than high–strength concrete with silica fume which replaces 5 to 20 percent of total cementitious material.

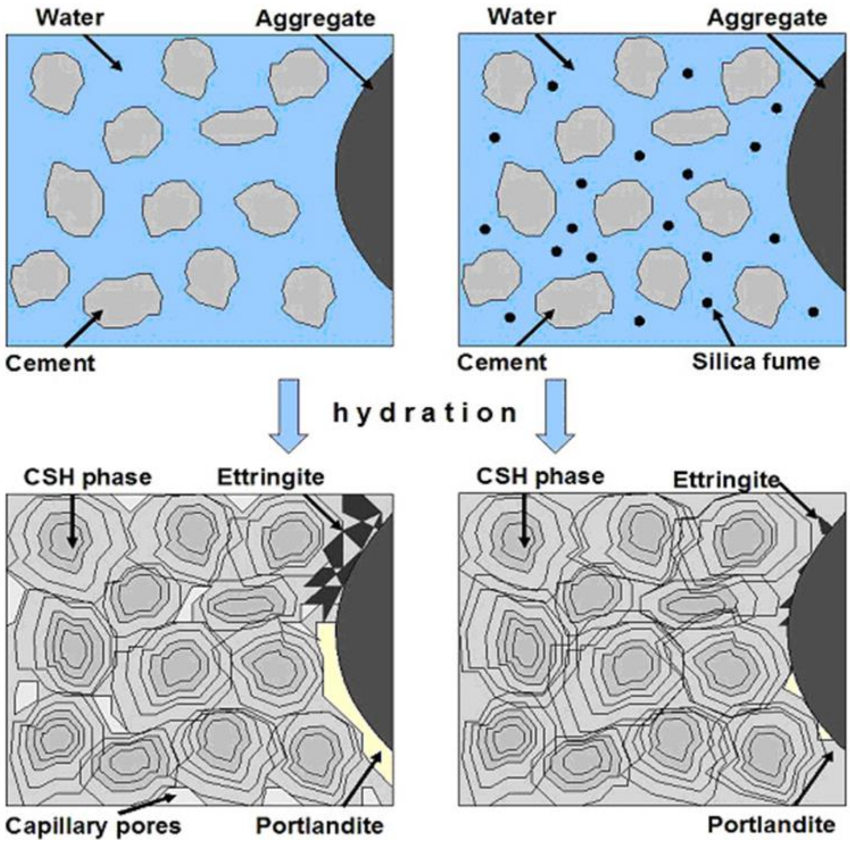


Figure 9 - Silica Fume as Micro–Filler, Decreases the Permeability and Increases Density, without Silica Fume (left), Concrete with Silica Fume (right) [36]

Silica fume is available in forms of raw powder, water based slurry, densified or palletized. [12] Silica fume in form of densified powder is the most common practice of adding silica directly to concrete. Water based silica fume is the most efficient in rapid disperse in concrete mixture; however such packaging are hard to maintain up until the moment of usage. However, raw powder and palletized silica fume are not common in high–strength production. Raw powder is very difficult to handle, and palatalized silica fume is rather used in production of blended hydraulic cement with silica fume. Thus, it is definite that silica fume improves concrete's resistance and durability; that it is also suitable for achieving higher strength during early ages, and that it is the main accelerator in widespread of high–strength concretes.

In contemporary concrete technology, chemical admixtures are indispensable constituent materials. Whether discuss about conventional concrete or high–strength concrete, chemical admixtures have an important role in the production of concrete. Focusing on high–strength concrete, its production would be impossible without superplasticizers such as high–range water reducers, retarders etc. As SCM (supplementary cementitious materials/mineral admixtures), chemical admixtures improve both fresh and hardened concrete. Without chemical admixtures, even the ability for transport, placement and curing of conventional normal strength concrete would be questionable, and therefore lack of chemical admixture in high–strength concrete would make high–strength concrete impossible. [12]

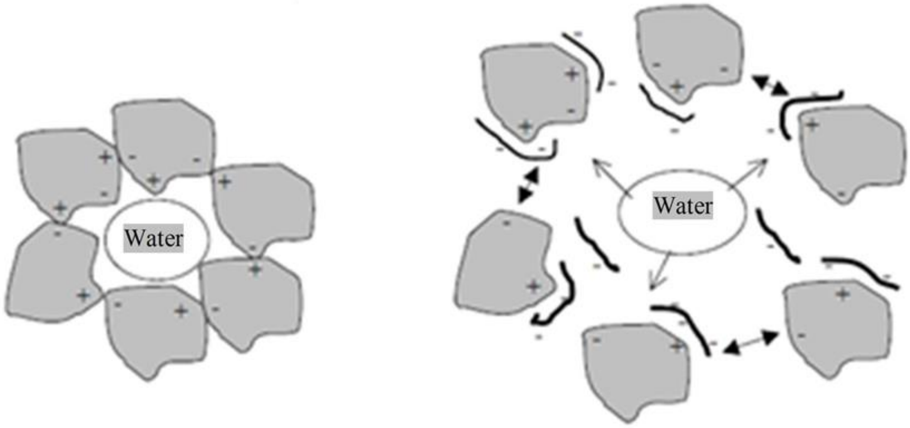


Figure 10 - Cement Particles and Water Reaction without Superplasticizer – HRWR (left) and Cement and Water Particles Reaction with Superplasticizer – HRWR (right) [37]

Possible reaction between chemical admixtures and cementitious materials and mineral admixtures may decrease the efficiency of chemical admixtures. Efficiency of chemical admixtures may also be decreased due to aggregate shape, grading, water content, slump, mixing time, temperature of curing etc. Quantity of specific chemical admixture relies on the amount of cement or cementitious material. With high–strength concrete, the most important chemical admixture is high–water reducing admixture; however it is not uncommon to use retarders, hydration stabilizers, viscosity modifiers and accelerators.

High–range water reducing admixtures as more common superplasticizers are generally classified in four categories: sulfonate melamine–formaldehyde condensates, sulfonate naphthalene–formaldehyde condensate, modified lignosulfonate and poly–carboxylate derivatives. These linear polymers' contain sulfonic acid groups attached to the polymer back bone at regular intervals, providing high performance at early ages.

High–range water admixtures (HRWR) decrease W/C ratio, but it is important to determine correct dose and type of the admixture. Thus, HRWR increase strength, with decrease W/C ratio, while maintaining slump constant, but also increase slump while maintaining W/C ratio. However, if there is a concern about conventional water reducing admixtures, principle remains the same with a bit lower influences.

The highest percentage of concrete's volume goes to aggregate volume. Selection of the appropriate aggregate is very important; in high–strength concrete, the best quality and the strongest aggregates are required. What effects aggregate is its density, grain size composition, shape and texture of the aggregate surfaces. [12]

For instance, inappropriate use of aggregates, or use of low strength aggregates in high–strength concrete generally results in weaker final material than it would be if appropriate aggregate were used, which generally results with higher quality; the rest of the constituent materials remain constant. Aggregates that are suitable for conventional normal strength concrete may not strictly be appropriate for high–strength concrete.

In addition, conventional normal strength concrete may use, aggregates are usually stiffer and are not considered weak points. Possible cracking, failures and fractures are generally happening in cement paste. However, in case of high–strength concrete addition of fly ash, silica fume or other mineral admixture increases the strength and stiffness of cement paste. Therefore, weak point of concrete mixture is transferred to the aggregates and indicates necessity for aggregates of the highest quality.

In high–strength concrete, when the volume of fine cementitious material increases, the necessity to use fine aggregates such as sands (less than 0.25 mm in diameter) decreases when compared to normal strength concrete. On the other hand, use of fine aggregates varying from 0.5 to 4 mm in diameter is desirable for high–strength concrete as long as it increases concrete's workability. With more strengthened and stiffen cement paste, crucial weak points in strength of high–strength concrete become coarse aggregates, and also the bond between cement pastes and aggregates larger than 4 mm in diameter. With increased quantity of coarse aggregate surface area of bond between cement paste and aggregate increases respectively, which results in increase of weak points. Thus, smaller aggregate grading, generally up to 16 mm in diameter become more workable as the desired strength increases. Factors which determine quality of aggregate besides the strength are surface texture, aggregate shape, grading, cleanliness and aggregate mineralogy.

In high– strength concrete rough textured and angular aggregates increase mechanical cement paste–aggregate bond and therefore such aggregates are more workable in high–strength concrete. Any organic substance, dust or clay, must be removed from the aggregate in order to exclude possibility of any impurities during the production. Trap rock, granite, dolomite and quartzite are mineralogy types of aggregates, suitable for high–strength concrete.

Nowadays use of recycled water in concrete production increases, and also non–potable water is not uncommon in practice. However water with some kind of smell or any kind of colour or taste is considered as non-potable; rain water or water which is used to wash leftovers from concrete mixers is not suitable for production of high–strength concrete.

When water analysis shows that these types of water will not affect concrete's properties, application can be approved, although all recommendations advise on the use of potable water to exclude possibility of any impurities in water.

When compared to conventional normal strength concrete, W/C ratio in high–strength concretes is lower varying from 0.22 to 0.40. However, it is important to analyse whether the certain decrease in W/C ratio is necessary and whether it leads to the requested increase of concrete's strength and performance. [12]

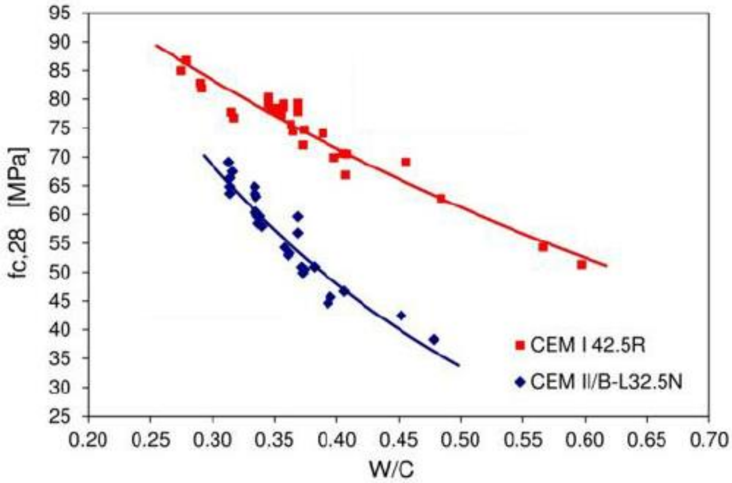


Figure 11 - Diagram Shows Inverse Ratios between Compressive Strength and W/C Ratio [15]

MIX DESIGN AND PROPORTIONING OF HIGH–STRENGTH CONCRETE (HSC)

Proportioning of constituent materials in high–strength concrete must be very precise and developed to the detail. Primary principles of high–strength concrete's proportioning are very different from principles commonly found in conventional concrete production. Therefore, it is important to emphasize that conventional normal strength concrete's proportioning principles are very little or even not applicable at all to the proportioning of high–strength concrete. Also, high–strength concrete requires advanced materials, when compared to those materials used for conventional concrete. Thus, another important point to understand that materials used for conventional concrete are not suitable for use in high–strength concrete.

Challenges in proportioning of high–strength concrete lays on uniform increase of all concrete's properties, not on emphasizing only one or two concrete's properties while underestimating other properties. Therefore, every development of the high–strength concrete's mixture proportioning starts with preparation work where concrete's desired properties are defined. Whether concrete properties are classified as mechanical, durability, thermal and constructability properties, or properties of fresh concrete and hardened concrete, it is important to declare what they are and what their presumed values are.

It is not wrong or incorrect to set up experimental studies which focus on reaching increased target strength or increased target resistance to the specific conditions. However, it is important for successful proportioning to look for mixture's durability, consistency, workability, setting time, early strength development etc.

As previously mentioned, high–strength concrete proportioning has been topic of many researches and experimental studies since the mid–20th century. However, mixture proportioning and mix designs that are already developed cannot be reused for different areas due to change in properties of the constituent materials of the concrete's mixtures. However, with accordance to those researches and experimental studies, there are some common rules which influence and increase the concrete's strength. These steps are: to reduce paste porosity, to reduce micro cracking at interfacial zone and micro cracking in the paste and increase the mixture's homogeneity which affects the concrete's strength.

Once, the desired concrete properties are defined, next step is to get closer with constituent materials and their proportions in the mixture. First, chosen constituent materials for high–strength concrete must be of the best quality. All mechanical and physical properties of the aggregate, cementitious materials and cement paste should be examined, and also target W/B ratio (Water/Binder ratio) should be presumed with accordance to target concrete's properties. Crucial factors in mixing proportioning of high–strength concrete which influence target characteristics are W/B ratio, interfacial transition zone and aggregate.

One of the most important factor which influence concrete's properties, strength and durability, is W/C ratio. W/C ratio emphasizes the problematic of capillary porosity which generally refers to the distance of cementitious particles at the time of the hardening of concrete mixture. Whether discuss conventional normal strength concrete or high–strength concrete, one of the basis of concrete's proportioning is actually relation between W/C ratio and strength.

Duff Abrams, was the first researcher, who has conducted many scientific experiments and researches on W/C ratio influences on concrete mixture during the period of four years. He published the book "*Design of Concrete Mixtures"*, [1] back in 1920, with the first ever definition of the inverse relationship between W/C ratio and concrete's strength.

As W/C ratio is one of the most important factor in concrete mixture design, we have to mention all possible synonyms used for W/C ratio, which depend on cementitious materials used in concrete mixture and addition of numerous admixtures. It is not uncommon to find synonyms such as: water/cementitious material ratio – W/CM ratio, water/binder ratio - W/B ratio, water/ cement + pozzolana ratio – W/C+P ratio, etc.

In high–strength concrete W/C ratios are commonly lower than 0.40. However, in proportioning and deciding of W/C ratio it is important to analyse fresh concrete's properties and behaviour. It is better to increase the W/C ratio a bit in order to have better performances, than force the lowest possible W/C ratio, which would result in poor and weak concrete.

High–strength concrete's overall performances is greatly affected by shape, size, grain size distribution and texture of coarse aggregate used in the mixture. Aggregate proportioning for mixture design generally requires more time to develop appropriate sizes, grain size distribution etc. Generally, high–strength concrete with higher strengths aggregates of lower maximum sizes is desirable. However, larger aggregates become very important for other mechanical properties.

Such divergence in finding the best aggregate, finally resulted in agreement that smaller size aggregates should be used for high–strength concrete with addition of admixtures that could make up for other mechanical properties.

Issues and proportioning principles of aggregate choice for high–strength concrete greatly explains why proportioning principles of normal strength concrete are not applicable to high–strength concretes. For instance, in conventional normal strength concrete, smaller aggregates decreases concrete's strength while at the same time use of smaller aggregates is more than desirable to achieve higher strengths. However it is important to notice that smaller aggregates state for aggregates with diameter of 0.5 to 4 mm, and coarse aggregates of 4 to 8 mm of grain size.

Another issue lays in the choice of shape and texture of the aggregate. Although river sand or uncrushed aggregate require lower rates of water, which could reflect on decreased W/C ratio, aggregates with such shape and texture are not very desirable for HSC due to weak bond achieved between cement paste and aggregate. On the other hand, crushed aggregates provide much better bond between aggregate's surfaces and cement paste. Thus, for high–strength concrete proportioning, crushed, clean and high quality aggregates are chosen.

In high–strength concrete, along with W/C ratio and aggregate, bond between cement paste and aggregate or interfacial transition zone greatly influences mechanical properties and durability.

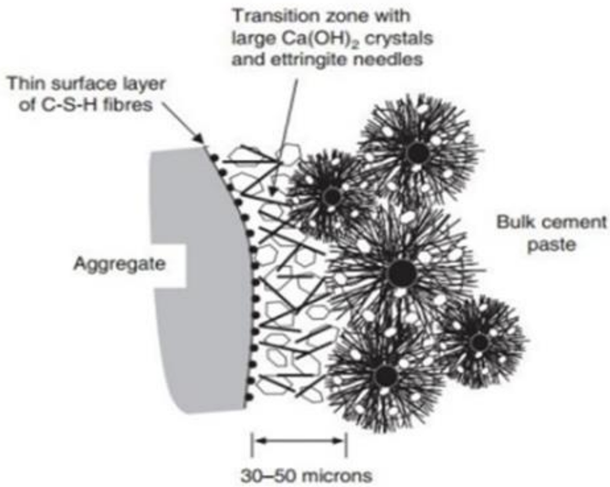


Figure 12 - Schematic View of the Aggregate and Cement Paste Bond through Interfacial Transition Zone, Representing Three Crucial Factors in High–Strength Concrete [54]

Addition of mineral admixtures, especially addition of silica fume improves properties of this specific bond. As the transition zone decreases, higher strengths are achieved. For instance, silica fume densifies matrix of interfacial zone, as this zone or bond gets denser the strength achieved increases. Thus, in high–strength concretes, and especially in silica fume high–strength concrete, cement paste–aggregate bond is very strong; cement paste is also very strong, and generally weak points of the high–strength concrete mixtures include cracking and crushing of the aggregate used.

PROPERTIES OF HIGH–STRENGTH CONCRETE (HSC)

Concrete is a composite material, mixture of aggregate and cement/ binder paste and mixture of raw materials (cement, chemical and mineral admixtures, water). Any of these constituent materials has its own mechanical, chemical and durability properties. As there is a variety of material types, there is also variety of possible mix designs. Therefore, it is very difficult to give strict and unique definition, literal or numeral for each of the concrete's properties. Important mechanical properties of high–strength concrete include compressive strength, modulus of elasticity, creep and shrinkage and durability related properties such as resistance to aggressive environments, chemical environment, fire resistance, cycling temperature changes, freeze–thaw resistance etc.

In the specific case of concrete, most properties depend on concrete's constituent materials' properties, however, interfacial transition zone is important in this case, or in other words, the bond between aggregate and cement paste.

The strength of concrete depends on properties of constituent materials, mix design of concrete or concrete proportioning, degree of hydration or curing temperature, rate of loading and method of testing. This means that quality and mineralogy of aggregate type, quality of cement and cementitious paste influence the strength of concrete.

High–strength concrete behaves differently from conventional normal strength concrete. For example high–strength concrete with reduced W/C ratio to 0.35 without adding silica fume at the 7th day of curing at the temperature of 20 ˚C gains 86 per cent of its target strength for the 28th day.

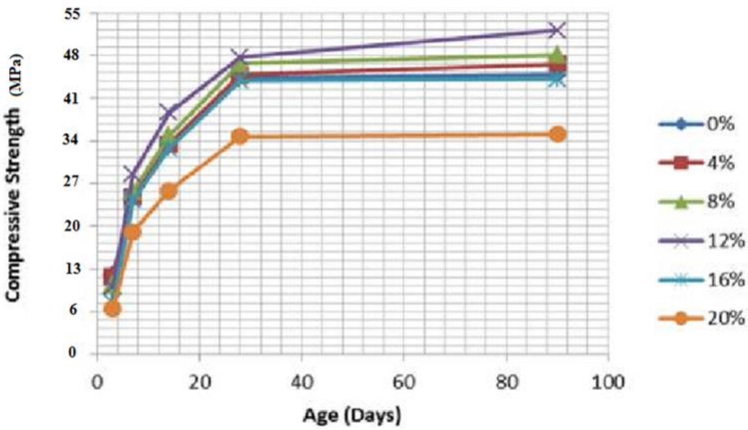


Figure 13 - Strength Development over Time of High–Strength Concrete with Different Percent of Silica Fume, Varying from 0 – 20 Percent [24]

When silica fume is added to the same mixture, and temperature increased to 50 ˚C, strength developed at the 7th day may reach 97 per cent of the strength expected at the

28th day. Thus, high curing temperature for high–strength concrete with silica fume results in earlier age of strength development. High–strength concrete tends to develop strength earlier, due to the use of supplementary cementitious materials (mineral admixtures), and curing temperature due to higher heat of hydration.

Thus, compressive strength is one of the high-strength concrete's main and most valuable properties. In the specific case of high–strength concrete, it is more related to the cement paste, which is directly related to the higher compressive strength.

Therefore, W/C ratio or W/B ratio, mechanical properties of aggregate, percentage of silica fume, fly ash or any other SCM, curing conditions, aggregate/ cement ratio, shape, texture of aggregate are all factors that influences concrete's compressive strength. However, it does not necessarily means that other properties of high–strength concrete mixture respectively increase with increase of strength.

Ratio of normal stress and a corresponding strain of tensile or compressive stresses defines modulus of elasticity. It is important factor and property of concrete, and very important in design for presumptions of deformations, particularly when related to construction of high–rise structures.

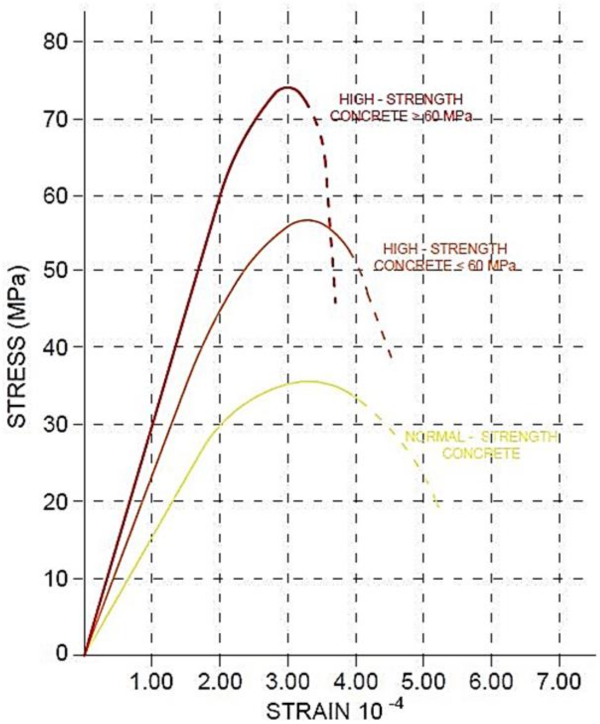


Figure 14 - Comparison of Stress–Strain Relationship for Normal Strength Concrete, High–Strength Concrete < 60MPa and High–Strength Concrete > 60MPa [53]

Modulus of elasticity increases with the use of aggregate larger in size; however the use of stiffener coarse aggregate with higher modulus of elasticity also increases concrete's modulus of elasticity. There are two elastic moduli, elastic modulus of aggregate and elastic modulus of cement paste.

Besides static modulus of elasticity, there is also a dynamic modulus of elasticity which corresponds to small instantaneous strains. It is generally 40 – 30 – 20 percent higher than static modulus of elasticity.

Shrinkage is defined as time dependent strain that occurs in absence of applied load. Shrinkage may occur in early and later age. Therefore, shrinkage is classified into three groups: plastic shrinkage, early–age shrinkage and later–age shrinkage.

High–strength concrete is more likely to undergo plastic shrinkage cracking because high–strength concrete bleeds at slower rate and has less bleed than conventional normal–strength concrete. So any cracking which may occur on the surface of fresh concrete after its placing is plastic shrinkage cracking. High–strength concrete slabs are vulnerable to plastic shrinkage due to excessive exposed surface where evaporated water can be replenished with bleed water. In order to prevent plastic shrinkage it is necessary to prevent dehydration. Long–term concrete's performance depends on properties developed at early ages of concrete; to provide long–term performance early age shrinkage must be controlled. However good choice of the material results in reduction of concrete's shrinkage. Quartz, dolomite, limestone, granite or feldspar are, for instance, aggregate types which may reduce shrinkage, while slate, basalt or trap rock lead to larger shrinkage.

On the other hand, time dependent strain under sustained loading is called creep. Creep is important where deflections or member shortenings must be eliminated. For high–strength concrete, creep generally increases with applied stress, and with stress – strength ratio of 0.6.

In the case of high–strength concrete, workability may become an issue if mixing proportions are not carefully designed. In order to have full performance and to be workable, high–strength concrete mixture has to be dense and void free. High–strength concrete mixture must be workable, easy to vibrate and must remain fluid in order to get around steel reinforcement. Slump testing generally shows the workability of concrete mixture. Slump test procedures in Europe are defined by EN 12350-2. [27] Loss of workability or slump may be repaired with addition of superplasticizers.

Durability is one of the most important properties of concrete, or any other material. Although the accent of the high–strength concrete is based on strength development, durability also requires high attention in order to respectively increase with strength.

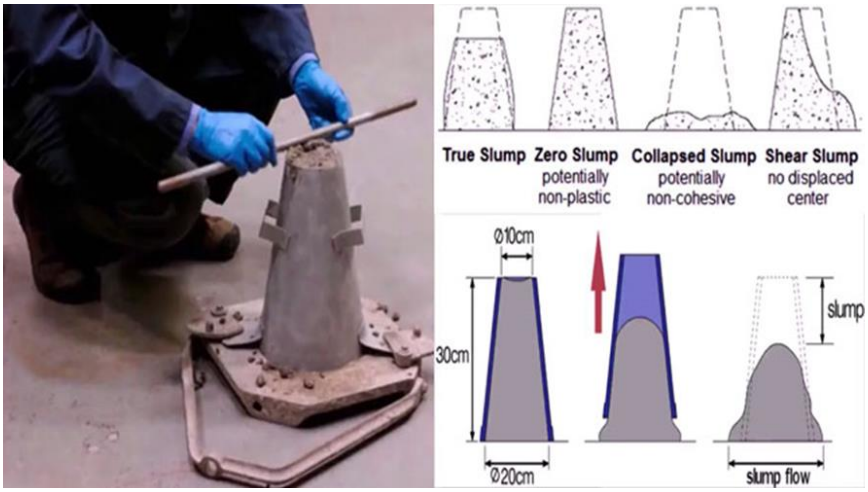


Figure 15 - Slump Test Procedure [50]

Concrete structures are commonly replaces, repaired or demolished due to durability related issues, and not the loss of strength. Most of the durability related issues are caused by infiltration of water, salts, or sulphate–bearing compounds which cause or initiate cracking, disintegrations etc. Reducing cavities and concrete permeability is an effective way to increase concrete's durability. Decrease of W/C ratio or W/B ratio decreases permeability, which leads to a conclusion that high–strength concrete's low W/C or W/B ratio increases durability of high–strength concrete.

When compared to conventional normal strength concrete, high–strength concrete is more uniform and homogeneous microstructure. In high–strength concrete, bulk matrix is very dense. As the material's micro structure becomes denser, the permeability reduces. Decreased permeability means improved resistance to freezing, thawing, sulphate and chloride penetration, chemical attacks etc. There are three types of permeability; permeability to gasses, permeability to liquids and permeability to chloride ions.

Compared to any other structural material, steel or wood, one of the greatest advantages of concrete as construction material is high fire resistance. Generally all concrete structures are capable to maintain load–bearing capacity when exposed to fire and that is why concrete's structures are marked as fire-resistant. However, high–strength concrete has some lower rates of fire resistance when compared to the normal strength concrete. Reduction of cross sections of structural elements as one of the high–strength concrete's advantage, is also the one to decreases its fire resistance.

Although high–strength concrete has lower fire resistance than normal strength concrete, it still has higher fire resistance than any other structural materials' and becomes economically efficient solution able to improve its fire resistance.

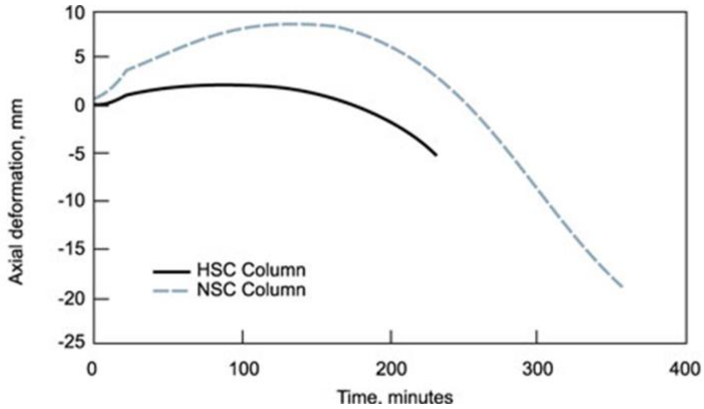


Figure 16 - Curves of Axial Deformation during Fire Exposure of High–Strength Concrete and Normal Strength Concrete [52]

In the specific case of high–strength concrete structures, another problem may occur within lower permeability; extremely high water pressure generated during fire exposure may not be able to escape through small cavities and canals, due to high density of concrete in order to lower inner temperatures. There are several ways to overcome this problem; the most efficient is the method in which synthetic fibre reinforcement and polypropylene is added. Polypropylene fibres in high–strength concrete melt at approximately 160 ˚C, leaving moisture channels with effective internal pressure control.

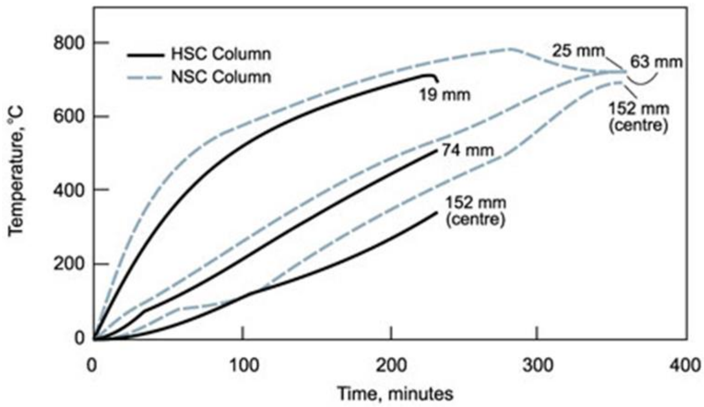


Figure 17 - Different Temperature Entry towards the Central Axis of the Columns Made of High–Strength Concrete and Normal Strength Concrete during the Standard Fire Exposure [52]

Chemical resistance of high–strength concrete refers to its ability to withstand any possible chemical reaction in between constituent materials in concrete mixtures, and to prevent any possible damages, cracking etc. To be more precise, chemical resistance is classified as resistance to alkali–silica reactions, sulphate resistance and corrosion resistance. By definition, high–strength concrete has more cementitious materials in the mixture proportioning when compared to conventional normal strength concrete. As long as the Portland cement is the carrier of the highest quantity of alkalis which initiates any of these reactions, high–strength concrete must be designed to withstand cracking or damages caused by alkali–silica reactions. On the other hand, the highest sulphate resistance is achieved with low W/B ratio, and therefore it is obvious that high–strength concrete is highly sulphate resistant material due to common use of low W/B ratios.

Corrosion resistance, however, is more concerned with reinforcement than concrete itself. However, corrosion resistance increases due to decreased concrete's permeability. Thus, high–strength concrete as a very low permeable material has very high rate of corrosion resistance.

Abrasion is one of the most common types of possible damages on concrete surfaces. Exposure of concrete structures to high temperature variations (freeze–thaw cycles), aggressive environment, or constant contact with water, generally result in dorms of abrasion damages. So, ability of concrete surface to resist being worn out by any source of friction, is called abrasion resistance. Aggregate properties, surface finish, liquid hardeners or inadequate curing all influence the rate of abrasion resistance of concrete.

Abrasion resistance as a property of structural material does not have high attention in structuring of buildings. Although if discuss bridges and dams, abrasion resistance plays an important role in choice of the structural material based on its capacities for resisting abrasion. Among structural materials, silica fume high–strength concrete proved to be as very efficient in resisting any forms of abrasion. Also, silica fume high–strength concrete with lower W/B ratio showed as very efficient in repairing the abrasion deteriorations.

Experimental studies conducted in 1994 on abrasion resistance of high–strength concrete with or without chemical and mineral admixtures, showed that abrasion resistance of high–strength concrete varies inversely due to W/B ratio, the paste volume of concrete and the porosity. This leads to a conclusion that high–strength concrete with mineral admixtures should also contain high range water reducer or superplasticizers in order to keep increasing its abrasion resistance. Otherwise, the use of the mineral admixtures without HRWR in concrete demands more water which instantly decreases abrasion resistance.

POSSIBILITY FOR PRODUCTION OF HIGH–STRENGTH CONCRETE (HSC) IN BOSNIA AND HERZEGOVINA EXPERIMENTAL STUDY

High-strength concrete has been used as structural material for several decades in the world. Worldwide, there are numerous concrete plants which routinely deal with production, transportation and casing of high–strength concrete in construction of both prefabricated and cast in situ structures. However, there is almost no concrete plant which produces high–strength concrete in Bosnia and Herzegovina, and if we examine experimental studies, there is also little or no tracks of concrete development in this direction. Thus, experimenting with high–strength concrete production in Bosnia and Herzegovina challenges capabilities and qualities of domestic materials.

Experimental part of this study is about concrete mixtures and production of high–strength concrete with available domestic materials in Bosnia and Herzegovina. This experimental study is shaped through the use of five different concrete mixtures; the first concrete mix was made as ordinary concrete consisting of aggregate, cement and water. The second one was basically the first mixture but with decreased W/C ratio and addition of superplasticizer. The last three mixtures were made with the addition of silica fume in 5, 10 and 15 percent of total cementitious content in mixture, in order to increase the possibility of achieving higher compressive strengths comparable with common worldwide workable mixtures of silica fume high–strength concrete achieved by using superplasticiser. For each concrete mix, three concrete specimens were produced for further testing of hardened concrete.

All materials used for the concrete mixtures examined in this study are domestic, and include aggregates from quarry "Džehveruša" near Cazin, cement of Heidelberg CEMENT Group in Kakanj, silica fume of BSI Metalleghe in Jajce, and water from water supply system and water source "Klokot" in Bihać. The only exception is superplasticizer which is a product of "SIKA Group", but also available on the market in Bosnia and Herzegovina.

Experimental study of this study focuses on compressive strength of hardened concrete, and all experimental phases are performed to meet EN 206 [33] and EN 12390 [29-32]. Since the goal is to produce high–strength concrete in Bosnia and Herzegovina out of domestic materials, this study aims to conclude whether this is possible or not with these specific materials. All experimental work is conducted in the laboratory of the Institute for Material Testing at the Faculty of Technical Studies, University of Bihać in Bosnia and Herzegovina

Cementitious material used for this study is Portland Cement CEM I 52. 5 N, produced in Cement Factory in Kakanj. Manufacturer, claims that Portland Cement (PC) CEM I 52. 5 N can be used as the cement for both plain concrete and reinforced concrete with high compressive strengths. This cement consists of 95 – 100% of Portland cement clinker, up to 5% of raw gypsum and minor additional substances. Presence of raw gypsum in PC CEM I 52.5 N, regulates setting time and accelerates it. CEM I 52.5 N is enables greater compressive strengths at both early and later ages of concrete. Specific gravity of the CEM I 52.5 N is 3.18, and its grain size is approximately in rage from 5 – 30 μm. [50]

Manufacturing process and product itself is coordinated with the EN 197 -1 [28] for cement, and is regularly inspected and approved by GIT Tuzla, Bosnia and Herzegovina and IGH Split, Croatia.

Table 1 - Chemical Properties of Portland Cement CEM I 52.5 N Heidelberg Cement Group (Kakanj) [43]

	CEM I 52.5 N	STANDARD
SO_3 (%)	2.60	≤ 4.0
Cl (%)	0.01	≤ 0.1

Table 2 - Physical and Mechanical Properties of Portland Cement CEM I 52.5 N Heidelberg Cement Group (Kakanj) [43]

PROPERTY	CEM I 52.5 N	STANDARD
Initial setting time (min)	130	≥ 45
Soundness (mm)	1.0	≤ 10
Compressive strength after 2 days (MPa)	27	≥ 20
Compressive strength after 28 days (MPa)	64	≥ 52.5 ≤ 72.5

Aggregate used for this study is crushed dolomite from the quarry "Džehveruša" near Cazin, Bosnia and Herzegovina. High quality aggregate of "Džehveruša" is so far used for infrastructural projects (roadways, streets, curbs etc.) and as constituent material for structural concretes. Quarry "Džehveruša" can separate aggregate sizes up to the 31.5 mm, while sizes 0 – 4, 4 – 8, 8 – 16, 16 – 31.5, that remains at the last sieve are reselected and taken for secondary breaking and blasting.

Figure 18 - Micro Location of Quarry "Džehveruša" Cazin, Bosnia and Herzegovina [44]

Grading percentages in the quarry are as follows: size 0 – 4 takes 50 per cent of total aggregate capacity of "Džehveruša", size 4 – 8 takes 25 per cent, grain size of 8 – 16 takes 20 per cent, and finally grain size group 16 – 31.5 takes only 5 per cent of total aggregate capacity of the quarry "Džehveruša". Maximum aggregate size used for high–strength mixtures is 16 mm. Thus, this experimental study includes grain size groups 0 – 4, 4 – 8, 8 – 16 mm.

Table 3 - Physical and Mechanical Properties of Aggregate Dolomite – Stone Pit Džehveruša (Cazin) [11]

PROPERTY	MEASURING VALUE
Compressive Strength of Air Dry Aggregate	129 MPa
Compressive Strength of Wet Aggregate	116 MPa
Compressive Strength After Freezing	114 MPa
Abrasion Value of the Aggregate	17.8 cm^3/ 50 cm ^{L}
Bulk Density	2839 kg/m^3
Specific Gravity	2.87
Porosity	1.73%
Water Absorption	0.44%
Impact Resistance	A = 16.15 % C = 22.6 %
Resistance to Crushing	21.0 %

Table 4 - Analysis of Aggregate Samples from Stone Pit Džehveruša (Cazin) [11]

	PROPERTIES	TEST METHOD	UNIT	GRADING			
				0-4	**4-8**	**8-16**	**16-31.5**
1.	*Grain size*	B.B3.100	Numerical grain size distribution is shown below.				
2.	*Content of grains below* *0 ,09 mm* *0.063 mm*	B.B8.036	%	*7.64* *7.02*	*1.16* *1.08*	*0.62* *0.58*	*0.29* *0.27*
3.	*Resistance to frost cycles*	B.B8.044	%	*2.98*	*0.98*	*0.72*	*1.02*
4.	*Suction*	B.B8.021	%	-	*0.52*	-	-
5.	*Grain shape* *3 : 1*	B.B8.048 B.B8049	%	- -	*17.62* *0.20*	*8.98* *0.21*	*5.41* *0.27*
6.	*Content of weak grains*	B.B8.037	%	-	*2.64*	*3.20*	*0.20*
7.	*Content of worn grains*	B.B8.037	%	-	*0*	*0*	*0*
8.	*Clay lumps*	B.B8.038	%	-	*0*	*0*	*0*
9.	*Organic impurities - colorimetric analysis*	B.B8.039	coloration	*Liquid lighter than standard*			
10.	*Content of crushed grains*	B.B8.004	%	-	*100*	*100*	*100*
11.	*Resistance to grinding and wearing down – Los Angeles Method*	JUS B.B8.045	coefficient LA	*gradation "A"* -------*18.82 %* *gradation "B"* -------*16.98 %*			
12.	*Crushing value*	U.M8.030	%	-	*15.88*	*22.87*	*24.17*
13.	*Sand equivalency*	U.M1.040	%	*83.59*	-	-	-
14.	*Fineness modulus:* *asphalt* *concrete*	B.B2.010	%	*2.76* *3.40*			

15.	*Heat resistance at 300˚C* *heat - shock with 700 ˚C*	% weight loss		*0.0* *0.1*			
17.	*Density: bulk dense*	B.B8.030	*kg/m³*	*1745* *1925*	*1570* *1710*	*1570* *1740*	*1560* *1780*
18.	*Specific weight*	B.B8.031	*kg/m³*	*2852*	*2812*	*2838*	*2868*
19.	*Mineralogical – petrographic analysis*	dolomite					

Indispensable part of this experimental study was aggregate analysis. Aggregate analysis in this study included determination of particle size distribution of aggregates. Sieving part in determining particle size distribution was done with accordance to EN 933 – Tests for geometrical properties of aggregates – Part 1 [34]. Part one explains sieving method and states that sieves must have squared meshes, of 0.063, 0.125, 0.250, 0.500, 1.00, 2.00, 4.00, 8.00, 16.00, 31.50, 63.00 and 125.00 size expressed in millimetres.

Figure 19 - Sieve Shaker for Aggregate Grain Size Separation

After the aggregate sample has been sieved, retains on each sieve are carefully taken out and separately weighted. Next step includes numerical analysis and calculation of passing through each sieve in percentages, which actually defines grain size distribution of the aggregate. Thus, what this analysis seeks for is determining the most efficient ratio between percentage parts of different grain sizes. In general, aggregate grain sizes for high–strength concrete are limited to 16 mm, and percentage part of grain size groups 0 – 4 and 8 – 16 takes higher percentages in aggregate composition than grain size group 4 – 8. The first aggregate sample designed for analysis, had percentage ratio of grain sizes of 33 – 27 – 40. Sieving process lasted for eight minutes, because one sieve size required one minute of shaking on average. Once the aggregate was separated to specific grain sizes, retains on each sieve were carefully taken out and separately weighted.

Table 5 – Sample 1 - Percentage of Each Grain Size Group

GRAIN SIZE	PERCENTAGE %
0-4	33
4-8	27
8-16	40

Table 6 - Sample 1 – Grain Size "M" – Particle Size Distribution – Sample Weight: 3.00 kg

SIEVE SIZE	RETAINS ON SIEVE (g)	%	PASSING THROUGH SIEVE (g)	%
16	0.00	0.00	3000.00	100.00
8	1200.00	40.00	1800.00	60.00
4	810.00	27.00	990.00	33.00
2	243.54	8.12	746.50	24.88
1	336.60	11.22	409.90	13.66
0.5	164.93	5.50	244.90	8.16
0.25	113.65	3.79	131.30	4.38
0	131.28	4.38	0.00	0.00

Table 7- Sample 1 - Analysis of Sieve Passing of Aggregate for Given Mix Design

Sieve size mm	0	0.25	0.5	1	2	4	8	16
Grain Size M	**0**	**4.38**	**8.16**	**13.66**	**24.88**	**33**	**60**	**100.00**
Upper limit	0	18.00	34.00	49.00	62.00	74.00	88.00	100.00
Bottom limit	0	3.00	7.00	12.00	21.00	36.00	60.00	100.00

After the numerical grain size distribution was calculated for grain size distribution curve, next step was to compare derived values with both upper and lower limit values. Values of limit curves for grain size distribution were recommended by EN 206, [33] which showed optimum size distribution of 0/16mm. Upper limit refers to, grain size composition, with more fine aggregates and fine particles (0 – 4), while lower limit means that in grain size composition, coarse aggregates take higher percentages (grain size: 8 – 16).

Figure below shows the recommended limit curves of optimum grain size distribution for grain sizes of 0/16 mm, as recommended by EN 206 [33]. Curves A16, B16 and C16 are limits for continued grain size distributions, while D16 refers to the discontinued grain size distributions. If derived percentage passing values are less than the lower limits, A16 curve for continued grain size distribution and D16 curve for discontinued grain size distribution, such grain size composition is not suitable for concrete mixtures because it will result in low plasticity, workability etc. Favourable grain size distribution curve is positioned between the limits of A16 and B16, while acceptable curve is positioned between B16 and C16 curve. On the other hand, if the derived grain size distribution curve has values higher than the upper limit, such mixture will require more water due to high content of fine aggregate sizes and fine particles. For this experiment, the first step was to compare derived grain sizes distribution values with the mentioned limits A16 – lower limit and C16 – upper limit.

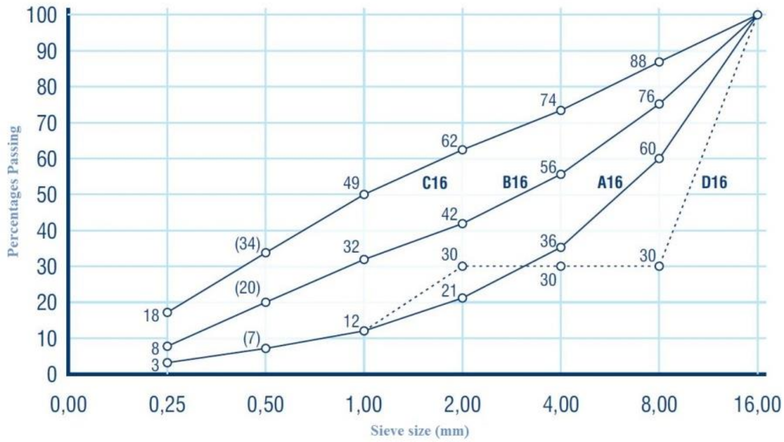


Figure 20 - Recommended Limit Curves of Optimum Grain Size Distribution for Grain Sizes of 0/16 mm [39]

Derived grain size distribution curve follows the bottom limit A16 curve, while breaking the limit at few points; therefore, such percentage part of 33 – 27 – 40 in grain size composition is not suitable for aggregate mixture.

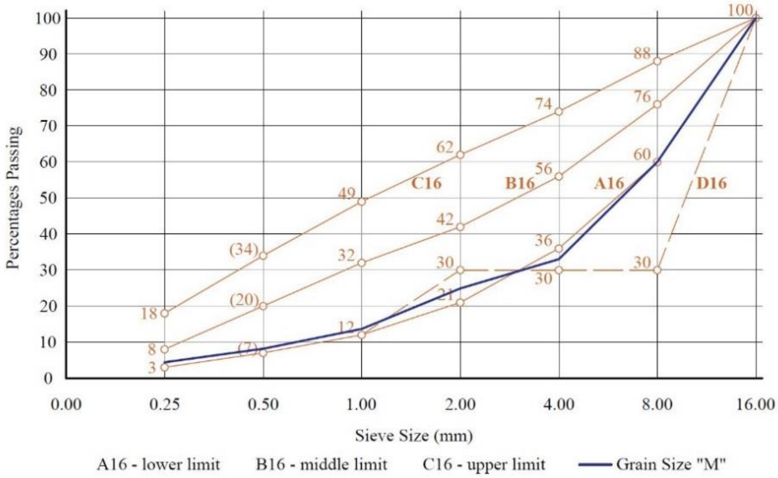


Figure 21 – Sample 1 - Derived Grain Size Distribution Curve Compared to Limits according to EN 206, Shows that Derived Curve is at the Bottom the Margin and Flows outside Margins

After unsuccessful determination of particle size distribution of the aggregate, next step was to approach the next particle (grain) size composition.

Table 8 – Sample 2 - Percentage of Each Grain Size Group

GRAIN SIZE	PERCENTAGE %
0-4	50
4-8	21
8-16	29

Table 9 – Sample 2 - Grain size "M" – Particle Size Distribution – Sample weight: 3.00 kg

SIEVE SIZE	RETAINS ON SIEVE (g)	%	PASSING THROUGH SIEVE (g)	%
16	0.00	0.00	3000.00	100.00
8	740.00	24.67	2253.60	75.12
4	749.00	24.97	1504.60	50.15
2	394.10	13.14	1110.50	37.02
1	497.40	16.58	613.10	20.44
0.5	265.90	8.86	347.20	11.57
0.25	116.70	5.39	185.50	6.18
0	185.50	6.18	0.00	0.00

Table 10 - Sample 2 - Analysis of Sieve Passing of Aggregate for Given Mix Design

Sieve size mm	0	0.25	0.5	1	2	4	8	16
Grain size "M"	**0**	**6.18**	**11.57**	**20.44**	**37.02**	**50.15**	**75.12**	**100.00**
Upper limit	0	18.00	34.00	49.00	62.00	74.00	88.00	100.00
Bottom limit	0	3.00	7.00	12.00	21.00	36.00	60.00	100.00

Second particle (grain) size composition with percentage parts of 0 – 4 = 50%, 4 – 8 = 21% and 8 – 16 = 29%, and the derived grain size distribution curve between upper (C16) and lower (A16) limits were acceptable for aggregate mixture.

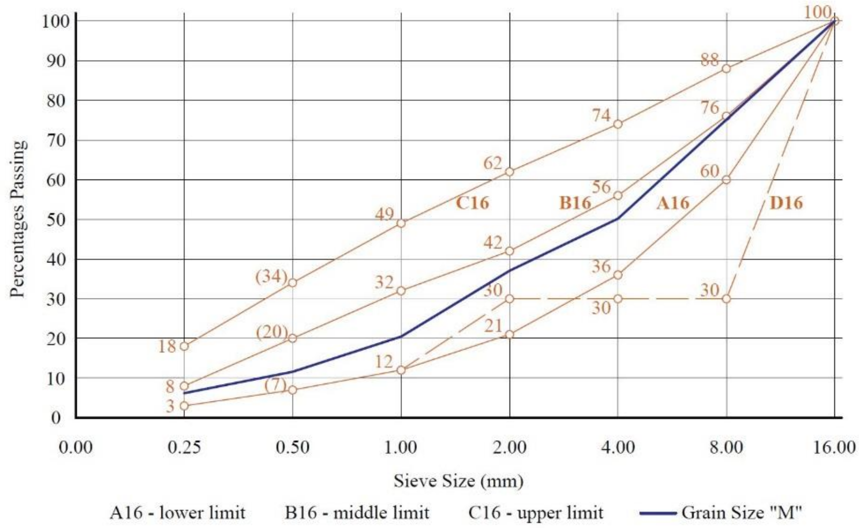


Figure 22 – Sample 2 - Derived Grain Size Distribution Curve Fit into Area between the Limits Recommended by EN 206

To see the actual success of the second particle (grain) size composition, percentage passing of the second aggregate sample was also compared to the middle limit – B 16. Table below shows numerical values of percentage passing of aggregate sample and middle limits recommended by EN 206, where it can easily be seen that this composition is actually creates desirable mix.

Table 11 - Sieve Passages of Aggregate Sample Compared to the Middle Limit B16 Recommended by EN 206

Sieve size mm	0	0.25	0.5	1	2	4	8	16
Grain size "M"	**0**	**6.18**	**11.57**	**20.44**	**37.02**	**50.15**	**75.12**	**100.00**
Middle limit – B16	0	8.00	20.00	32.00	42.00	56.00	76.00	100.00

Water used in this study is potable water from the Water Supply System in Bihać, from spring Klokot in Bihać. Its pH value is less than 7. This water is safe for human consumption, which means that it is clean and without any chemicals or other impurities that could bring undesired consequences for concrete mixtures, or develop harmful chemical reactions. Water is very important material in concrete mixtures, and it is a constituent which hydrate cement particles and creates binder paste. Water content in one cubic meter of concrete mixture may vary from 5 to 10 percent.

In high–strength concrete, water content is lower due to lower W/C ratios which are the practice in HSC proportioning. This study uses three different W/C ratios: first mixture uses 0.52, the following three use 0.35 and the last one uses 0.29. Lower W/C ratio means decreased water content in concrete mixtures, which leads to decreased workability of fresh concrete. With ultra–low W/C ratio, it becomes impossible to lubricate all cement grains and fine particles of aggregate. To overcome the problem of low workability, common practice is the addition of superplasticizer in order to increase concrete workability and to lubricate all cement grains, fine aggregate particles and silica fume particles in the case of silica fume high–strength concrete.

Table 12 - Common Relationship between W/C Ratio and Possible Concrete Compressive Strength [4]

CHARACTERISTIC COMPRESSIVE STRENGTH	**< 35 MPa**	**35–55 MPa**	**55 – 80 MPa**	**80 – 120 MPa**
W/C ratio	> 0.45	0.45 – 0.35	0.35 – 0.29	0.29 – 0.25
Chemical admixture	optional	WRA[2]/ HCA[3]	HCA	HRWR[4]
Mineral admixture	not necessary	not necessary	optional	silica fume

Superplasticizer used in this study is SIKA product, “Plastocrete N”. It is a brown liquid, chemical concrete admixture which is highly efficient plasticizer, lubricant and at the same time, a water–proofing agent. [47]

Advantage of “Plastocrete N” lies in improving concrete workability without increasing of the water presence in concrete mixture, what makes it excellent for concrete mixtures which require very low W/C ratio. Plastocrete’s advantage is also its ability to reduce shrinkage and increase both durability and strength.

[2] WRA – Water Reducing Admixture
[3] HCA – Hydration Controlling Admixture
[4] HRWR – High – Range Water Reducer

Figure 23 – Superplasticizer – "Plastocrete N"

"Plastocrete N" meets standard EN 934–2 [35], and it is compatible for use with "SIKAFUME" (silica fume – product of SIKA Group). Sika fume and silica fume from B.S.I. have the same chemical and physical properties, thus possibility of negative effect in combination of "Plastocrete N" and B.S.I. silica fume is excluded. "Plastocrete N" is dosed at 0.5 per cent of total mass of cementitious material used per cubic metre of concrete mixture. If "Plastocrete N" is overdosed, the use of retarders or air–entraining admixture may be required. [47]

Table 13 - Properties of "Plastocrete N" – Superplasticizer [47]

PROPERTY	MEASURING VALUE
Chemical base	Modified lignosulphonate
Density	1.07 ± 0.02 kg/l
pH value	6 – 10
Freezing point	-2˚C

Silica fume (microsilica) used for this study is a by–product of silicon and ferrosilicon industries, manufactured by B.S.I. d.o.o., member of Metalleghe Group and European Silica Fume Committee, in Jajce, Bosnia and Herzegovina. Chemical analysis, shows that silica fume from Jajce, contains 96.2 per cent of SiO_2, and is fully amorphous material. Such properties satisfy general requirements proposed by Silica Fume Committee in order for silica fume to be suitable in high–strength concrete production.

Figure 24 – Silica Fume

For this study, mineral admixture silica fume is added to Concrete Mix 3, Concrete Mix 4 and Concrete Mix 5 in 5, 10 and 15 per cent of total mass of cementitious material used per m^3.

Table 14 - Physical Properties of Silica Fume – Jajce [46]

PROPERTY	MEASURING VALUE
Particle size	0.5 µm
Bulk density	350 kg/m^3
Specific gravity	2.2
Specific surface	19.600 m^2/kg

As earlier highlighted, five different concrete mix designs were prepared in this study.

Mix design 1 is concrete mixture of cement, aggregate and water, with W/C ratio of 0.52 as used for ordinary concrete.

Figure 25 - Constituents for Experimental Mixture for Three Specimens of Mix Design 1

Total content of aggregate and cement required per cubic metre used for this mixture is also kept for the following four mix designs; also the ratio between fine aggregate (0 – 4) and coarse aggregate (4– 8 and 8 – 16) remains unchanged.

Table 15 - Mix Design 1 – Content of Constituents per m^3 and for Experimental Mixture

CONSTITUENTS	per $1m^3$	Trial mixture 3 cube specimens 15 cm x 15 cm x15 cm
Cement	450.00 kg	5.92 kg
Water	234.00 l	3.08 l
W/C ratio	0.52	0.52
Aggregate 0 – 4 mm	940.50 kg	12.78 kg
Aggregate 4 – 8 mm	395.00 kg	5.37 kg
Aggregate 8 – 16 mm	545.50 kg	7.41 kg

Mix design 2 is a concrete mixture of cement, aggregate and water, with the use of superplasticizer “Plastocrete N”. Superplasticizer is used in order to decrease water demand.

Figure 26 - Constituents for Experimental Mixture for Three Specimens of Mix Design 2

Water content is decreased by 30%, which respectively decreased W/C ratio to 0.35. The use of superplasticizer and decrease in W/C ratio is supposed to increase characteristic compressive strength.

Table 16 - Mix Design 2 – Content of Constituents per m^3 and for Experimental Mixture

CONSTITUENTS	per $1m^3$	Trial mixture 3 cube specimens 15 cm x 15 cm x 15 cm
Cement	450.00 kg	5.92 kg
Water	157.50 l	2.07 l
W/C ratio	0.35	0.35
Aggregate 0 – 4 mm	940.50 kg	12.78 kg
Aggregate 4 – 8 mm	395.00 kg	5.37 kg
Aggregate 8 – 16 mm	545.50 kg	7.41 kg
Superplasticiser – 0.5%	22.5 kg	0.023 kg

Mix design 3 is a concrete mixture of cement, aggregate, water and superplasticizer, with the addition of silica fume of 5 percent of total mass of cementitious material used per cubic metre.

Figure 27 - Constituents for Experimental Mixture for Three Specimens of Mix Design 3

With concrete mixture containing silica fume, more adequate term is W/B (water/binder) ratio or W/C + SF (water/cement + silica fume) ratio rather than W/C (water/cement) ratio. In this specific case, W/C+SF is kept 0.35. Thus, water content increased when compared to previous mix designs due to addition of silica fume.

Table 17 - Mix Design 3 – Content of Constituents per m^3 and for Experimental Mixture

CONSTITUENTS	per $1m^3$	Trial mixture 3 cube specimens 15 cm x 15 cm x 15cm
Cement	450.00 kg	5.92 kg
Water	164.00 l	2.16 l
W/C+SF ratio	0.35	0.35
Aggregate 0 – 4 mm	940.50 kg	12.78 kg
Aggregate 4 – 8 mm	395.00 kg	5.37 kg
Aggregate 8 – 16 mm	545.50 kg	7.41 kg
Superplasticizer – 0.5%	2.25 kg	0.023 kg
Silica Fume – 5%	22.5 kg	0.290 kg

Mix design 4 is a concrete mixture of cement, aggregate, water and superplasticizer, with the addition of silica fume of 10 percent of total mass of cementitious material used per cubic meter.

Figure 28 - Constituents for Experimental Mixture for Three Specimens of Mix Design 4

W/C+SF is kept at 0.35. Thus, water content is increased when compared to previous mix designs due to the increase of silica fume content.

Table 18 - Mix Design 4 – Content of Constituents per m^3 and for Experimental Mixture

CONSTITUENTS	per 1m^3	Trial mixtures 3 cube specimens 15 cm x 15 cm x 15 cm
Cement	450 kg	5.92 kg
Water	164 l	2.16 l
W/C+SF ratio	0.35	0.35
Aggregate 0 – 4 mm	940.50 kg	12.78 kg
Aggregate 4 – 8 mm	395.00 kg	5.37 kg
Aggregate 8 – 16 mm	545.50 kg	7.41 kg
Superplasticizer – 0.5%	2.25 kg	0.023 kg
Silica Fume – 10%	45 kg	0.590 kg

Mix design 5 is a concrete mixture of cement, aggregate, water, superplasticizer with addition of silica fume of 15 percent of total mass of cementitious material used per cubic meter.

Figure 29 - Constituents for Experimental Mixture for Three Specimens of Mix Design 5

This mixture is designed with the lowest W/C+SF ratio by decreasing water content and increasing percentage of silica fume. Thus, designed W / C+SF ratio is 0.29.

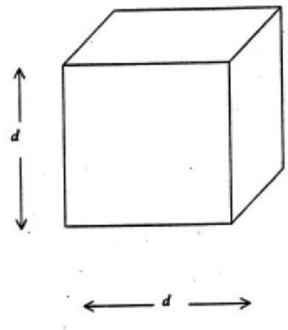
Table 19 - Mix Design 5 – Content of Constituents per m^3 and for Experimental Mixture

CONSTITUENTS	per $1m^3$	Trial mixtures 3 cube specimens 15 cm x 15 cm x 15cm
Cement	450.00 kg	5.92 kg
Water	150.10 l	1.98 l
W/C+SF ratio	0.29	0.29
Aggregate 0 – 4 mm	940.50 kg	12.78 kg
Aggregate 4 – 8 mm	395.00 kg	5.37 kg
Aggregate 8 – 16 mm	545.50 kg	7.41 kg
Superplasticizer – 0.5%	2.25 kg	0.023 kg
Silica Fume – 10%	67.5 kg	0.880 kg

Production of concrete specimens for this study was done in accordance to the EN 12390-2 [30]. All selected materials, or constituents for these experimental concrete specimens are approved by European standards; if not approved by licence, their properties are in accordance with standard requirements for constituents.

More specific specimens shapes, dimensions and other requirements or moulds are prescribed by EN 12390–1– Testing hardened concrete, Part 1 [29], while production and curing of specimens for strength tests were subject of EN 12390–2–Testing hardened concrete, Part 2 [30].

Thus, in accordance to EN 12390 -1, test specimen may be a cube, a cylinder or a prism, while the basic dimension *d* should be chosen to be at least three half of the nominal size of the aggregate used in concrete. [29]



d, mm	100	150	200	250	300

Figure 30 - Cube Nominal Sizes [29]

In this study, specimen is in a shape of a cube, with nominal size of 150 x 150 x 150 mm. Therefore, standard tolerances and requirements for cube specimens are as follows:

- Between molded surfaces, tolerance on the designated size (*d*) should be less than ±0.5 %;
- Between the top troweled face and the molded bottom face, the tolerance on the designated size should be less than ±1.0 %;
- The tolerance on the flat side of the potential load-bearing surfaces should be less than ±0.000 6*d*, in mm;
- The tolerance on the perpendicularity of the sides of the cube, with reference to the base, as cast, should be less than 0.5 mm. [29]

Moulds for concrete specimens should be watertight and non–absorbent. In order to achieve better water tightness, joints of moulds can be coated with wax, oil or grease. Moulds may be of any material which is suitable for producing concrete specimens. Generally moulds are made of steel or cast–iron, while plastic moulds are less used. Standard requirements for moulds for cubic specimens as follows:

- The tolerance on the designated size (d) of an assembled mould is ±0.25 %;
- The tolerance on the flatness of the four side faces of the mould should be ± 0.000 3d, for new moulds and ± 0.000 5d for moulds in use;
- The tolerance on the flatness of the top surface of the baseplate of the mould should be ± 0.000 6d for new moulds and ± 0.001 0d for moulds in use.
- The tolerance on the perpendicularity of the sides of a mould with respect to the adjacent sides and of the sides in relation to the base should be ± 0.5 mm. [29]

Moulds used for this study were steel moulds, calibrated without any inaccuracy for dimensions, flatness or perpendicularly, and one new unused plastic mould.

Mixtures were prepared one at the time, and cast immediately after the preparation. Before casting the concrete, mould's inner sides were covered with a thin film of non–reactive release material for better and easier removal of the mould from the hardened specimens. Concrete specimens should be cast into moulds by layers, where each layer should be compacted and should not exceed 100 mm of thickness.

Figure 31 - Example of Concrete Casting into the Mould and Hand Compaction with Rod in Layers

In accordance to EN 12390–2 [30] there are two ways of compacting the concrete specimens: through mechanical vibration and hand compaction.

All concrete specimens in this experimental study were compacted in 3 layers, with approximately 5 cm thickness. Each layer was first compacted by hand, using steel rod. Each concrete layer was subjected to 30 strokes.

Due to very low water content used for better compaction, each specimen is vibrated at the sieve shaker just enough to achieve the smooth top surface of the specimen, and to exclude any possibility of over vibrating.

All specimens after the compaction were left for 24 hours in moulds, at the temperature of 20 ˚C to harden. After 24 hours, specimens of the first mixture "Mix design 1" and other mixtures that used 0.5 per cent of superplasticizer were hardened and ready for further curing.

Figure 32 - Specimens after the Compaction were Left for 24 Hours in Moulds

According to EN 12390-2, curing may take place fully under water or in chamber with ≥ 95 percent of humidity and constant temperature of 20 ˚C ± 2 ˚C, which proved to be very hard to achieve. For this experimental study, all specimens were marked and submerged into clean potable water and left curing for the next 28 days.

According to European standard for concrete specification, performance, production and conformity – EN 206: 2014, compressive strength should be expressed as $f_{c,cyl}$ for cylindrical specimen and $f_{c,cube}$ for cubical specimen. Annex B of EN 206 states identifying criteria for compressive strength at the table B.1. Unless it is specified, otherwise specimens are tested at 28 day of curing, according to the EN 206. However, it is possible to test specimens for compressive strength earlier or later than 28 days.

Figure 33 - Curing of Concrete Specimens Fully under the Water

As previously mentioned, hardened concrete testing is standardized by EN 12390. EN 12390 – Testing hardened concrete – Part 3: Compressive strength of test specimens [31] states that specimens are loaded to failure in a compression testing machines conforming EN 12390 – 4 [32].

Table 20 - Identify Criteria for Compressive Strength [33]

Number *n* of test results for compressive strength from the defined volume of concrete	**CRITERION 1**	**CRITERION 2**
	Mean of n results (f_{cm}) N/mm^2	*Any individual test result (f_{ci}) N/mm^2*
1	Not applicable	$\geq f_{ck} - 4$
2 to 4	$\geq f_{ck} + 1$	$\geq f_{ck} - 4$
5 to 6	$\geq f_{ck} + 2$	$\geq f_{ck} - 4$

According to EN 12390 – 3 procedure of testing hardened concrete requires:

- *Specimen preparation and positioning* – Excess moisture from the specimen should be wiped before placing it in the testing machine. Also, testing machine surfaces should be clean and wiped. The cube specimen should be positioned so the load applied is perpendicular to the direction of casting. Specimen is centred with respect to the lower platen to an accuracy of ± 1% of the designated size of cubic or designated diameter of cylindrical specimens;

- *Loading* – As loading is selected constant rate of loading within 0.2 MPa/s (N/mm^2 x s) to 1.0 MPa/s (N/mm^2 x s). Load should be applied to the specimen without shock and increase continuously, at the selected constant rate ± 10%, until no greater load can be sustained. Maximum applied load is recorded and is used for identifying compressive strength;

- *Assessment of type of failure* – Examples of the failure of specimen shows that the tests were satisfactory and examples of unsatisfactory cubes are shown in the figure below. Unsatisfactory failures can be caused by, insufficient attention to the testing procedures, while the most common mistakes are made with positioning of the specimen, or another cause may be the testing machine faults. [31]

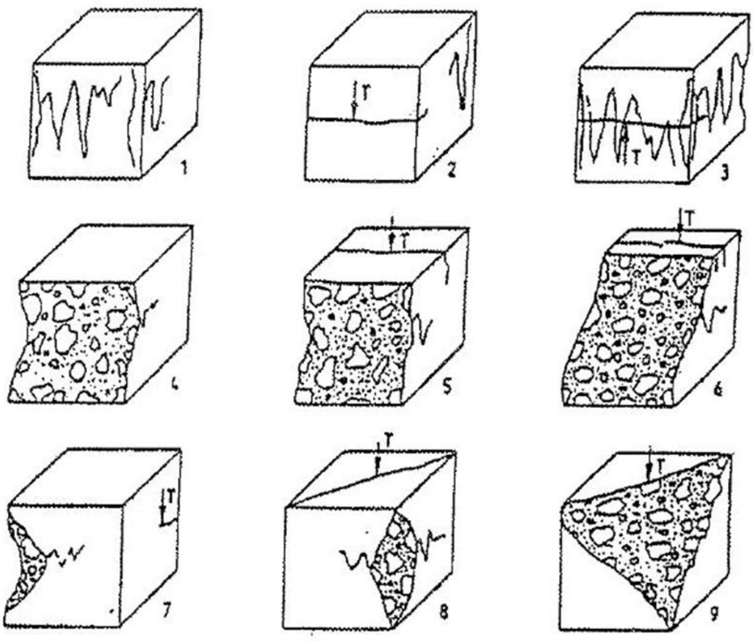


Figure 34 - Unsatisfactory Failures of Cube Specimens according to EN 12390-3 [31]

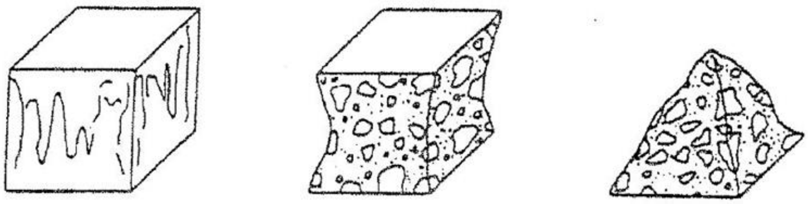


Figure 35 - *Satisfactory Failures of Cube Specimens according to EN 12390-3* [31]

- *Expression of results* – Compression strength is shown by the equatio*n:*

$$f_c = \frac{\mathrm{F}}{A_c} \tag{1}$$

where:

f_c - is the compressive strength in MPa (N/mm^2);

F - is maximum load to failure in N;

A_c - is the cross sectional area of the specimen on which the compressive force acts in mm^2.

Concrete strength was identified by compression testing machine. After the specimens were taken out of the water, they were wiped in order to remove the surface water. Before placing specimen at the compression machine, specifications including mass, dimension and geometry of each specimen were recorded and checked for any potential inaccuracies.

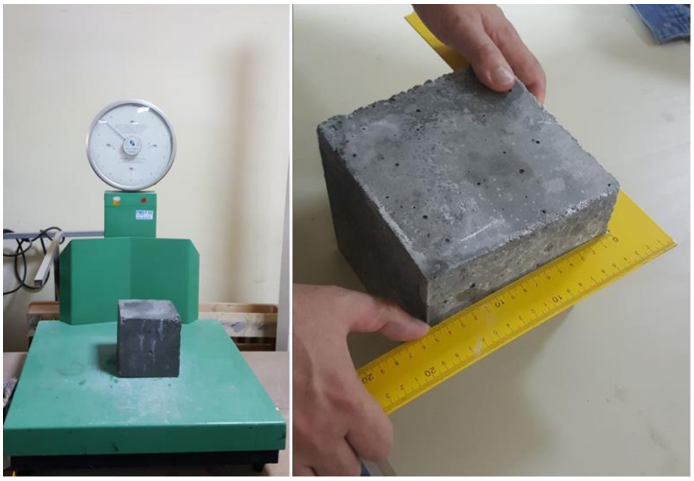


Figure 36 - Determination of Specimens Mass, and Checking for Dimensions and Geometry before Testing

After they were checked, specimens were carefully positioned at the centre of the compression testing machine, in order to prevent possible unsatisfactory failures of cube specimens.

Figure 37 - Compression Testing Machine

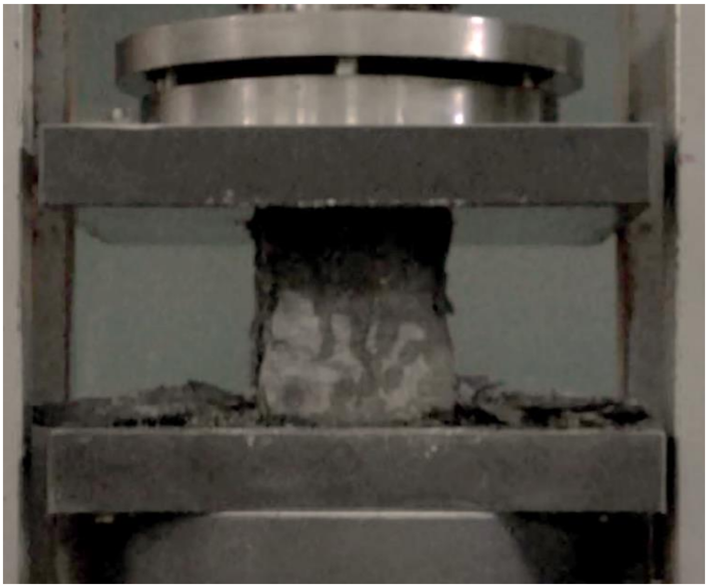


Figure 38 - Concrete Cube Specimen at the Moment of Failure at the Compression Testing Machine

Figure 39, shows common shape of test cube specimen failure in testing of hardened concrete at the compression testing machine. Best way to explain cracking and failure of specimens at the peak of applied loading is to state that four exposed sides were cracking uniformly, while top and bottom sides remained with small or no cracking and damages.

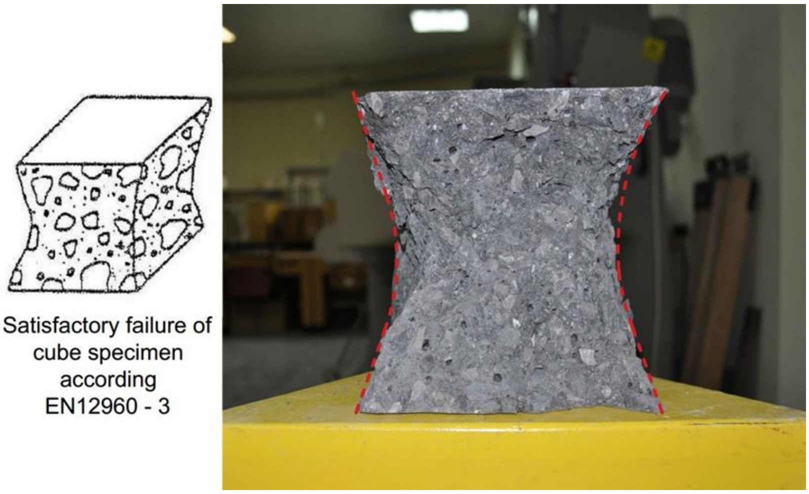


Figure 39 - Example of Satisfactory Failure of the Cube Specimen according to EN12960 – 3 (left) and the Actual Failure of One Cube Specimen from Study (right)

Figure 40 - Testing Results of Specimens for Concrete Mix Design 5

During the testing, the following results on concrete compression strength were obtained:

<u>Mix Design 1:</u> cement, water, aggregate

Date of casting: 8th August 2017

Date of testing: 5th September 2017

Table 21 - Results of Testing Hardened Concrete Specimens of Mix Design 1

MIX DESIGN 1	Age of specimen	Mass of specimen	Area	Maximum force at the failure	Compressive strength
	days	kg	mm^2	kN	MPa
Specimen 1	28	8.580	22500	1110	49.33
Specimen 2	28	8.520	22500	1020	45.33
Specimen 3	28	8.460	22500	1100	48.89
Mean Value		8.520		1076.67	47.85

According to the criteria stated in BAS EN 206:2014, Annex B, Table B1, Mix Design 1 is classified as concrete class C 35/45. As concrete specimens are produced in cubic shape, for determination of concrete class characteristic compressive strength (f_{ck}) value is 45 MPa.

BAS EN 206:2014, Annex B, Table B1, states that individual test result f_{ci} must be equal or greater than $f_{ck} - 4$, therefore:

- Specimen 1 - 49.33 MPa ≥ 41 MPa
- Specimen 2 - 45.33 MPa ≥ 41 MPa
- Specimen 3 - 48.89 MPa ≥ 41 MPa.

And mean value (f_{cm}) must be equal or greater than $f_{ck} + 1$

- Mean value 47.85 MPa ≥ 46 MPa.

<u>Mix Design 2:</u> cement, water, aggregate, superplasticizer 0.5%

Date of casting: 22nd August 2017

Date of testing: 19th September 2017

Table 22 - Results of Testing Hardened Concrete Specimens of Mix Design 2

MIX DESIGN 2	Age of specimen	Mass of specimen	Area	Maximum force at the failure	Compressive strength
	days	kg	mm^2	kN	MPa
Specimen 1	28	8.730	22500	1424	63,30
Specimen 2	28	8.730	22500	1370	60.89
Specimen 3	28	8.610	22500	1275	56.67
Mean Value		8.690		1356.33	60.30

According to the criteria stated in BAS EN 206:2014, Annex B, Table B1, Mix Design 2 is classified as concrete class C 45/55. For determination of concrete class characteristic compressive strength (f_{ck}) value is 55 MPa.

BAS EN 206:2014, Annex B, Table B1, states that individual test result f_{ci} must be equal or greater than $f_{ck} – 4$, therefore:

- Specimen 1 - 60.30 MPa ≥ 51 MPa
- Specimen 2 - 60.89 MPa ≥ 51 MPa
- Specimen 3 - 56.67 MPa ≥ 51 MPa.

And mean value (f_{cm}) must be equal or greater than $f_{ck} + 1$

- Mean value 60.30 MPa ≥ 56 MPa.

Mix Design 3: cement, water, aggregate, superplasticizer 0.5% and SF 5%

Date of casting: 22nd August 2017

Date of testing: 19th September 2017

Table 23 - Results of Testing Hardened Concrete Specimens of Mix Design 3

MIX DESIGN 3	Age of specimen	Mass of specimen	Area	Maximum force at the failure	Compressive strength
	days	kg	mm^2	kN	MPa
Specimen 1	28	8.675	22500	1540	68.44
Specimen 2	28	8.610	22500	1610	71.55

Specimen 3	28	8.730	22500	1588	70.58
Mean Value		8.672		1579.33	70.20

According to the criteria stated in BAS EN 206:2014, Annex B, Table B1, Mix Design 3 is classified as concrete class C 55/67. As concrete specimens are produced in cubic shape, for determination of concrete class characteristic compressive strength (f_{ck}) value is 67 MPa.

BAS EN 206:2014, Annex B, Table B1, states that individual test result f_{ci} must be equal or greater than $f_{ck} - 4$, therefore:

- Specimen 1 - 68.44 MPa ≥ 63MPa
- Specimen 2 - 71.55 MPa ≥ 63 MPa
- Specimen 3 - 70.58 MPa ≥ 63 MPa.

And mean value (f_{cm}) must be equal or greater than $f_{ck} + 1$

- Mean value 70.20 MPa ≥ 68 MPa.

<u>Mix Design 4:</u> cement, water, aggregate, superplasticizer 0.5% and SF 10%

Date of casting: 22nd August 2017

Date of testing: 19th September 2017

Table 24 - Results of Testing Hardened Concrete Specimens of Mix Design 4

MIX DESIGN 4	Age of specimen	Mass of specimen	Area	Maximum force at the failure	Compressive strength
	days	**kg**	**mm²**	**kN**	**MPa**
Specimen 1	28	8.600	22500	1470	65.33
Specimen 2	28	8.600	22500	1217	54.10
Specimen 3	28	8.500	22500	1413	62.80
Mean Value		8.570		1366.70	60.74

According to the criteria stated in BAS EN 206:2014, Annex B, Table B1, Mix Design 4 is classified as concrete class C 45/55. As concrete specimens are produced in cubic shape, for determination of concrete class characteristic compressive strength (f_{ck}) value is 55 MPa.

BAS EN 206:2014, Annex B, Table B1, states that individual test result f_{ci} must be equal or greater than $f_{ck} – 4$, therefore:

- Specimen 1 - 65.33 MPa ≥ 51MPa
- Specimen 2 - 54.10 MPa ≥ 51 MPa
- Specimen 3 - 62.80 MPa ≥ 51 MPa.

And mean value (f_{cm}) must be equal or greater than $f_{ck} + 1$

- Mean value 60.74 MPa ≥ 56 MPa.

<u>Mix Design 5:</u> cement, water, aggregate, superplasticizer 0.5% and SF 15%

Date of casting: 22nd August 2017

Date of testing: 19th September 2017

Table 25 - Results of Testing Hardened Concrete Specimens of Mix Design 5

MIX DESIGN 5	Age of specimen	Mass of specimen	Area	Maximum force at the failure	Compressive strength
	days	kg	mm^2	kN	MPa
Specimen 1	28	8.620	22500	1926	85.56
Specimen 2	28	8.610	22500	1450	64.40
Specimen 3	28	8.630	22500	1818	80.80
Mean Value		8.620		1731	76.92

According to the criteria stated in BAS EN 206:2014, Annex B, Table B1, Mix Design 5 is classified as concrete class C 55/67. As concrete specimens are produced in cubic shape, for determination of concrete class characteristic compressive strength (f_{ck}) value is 67 MPa.

BAS EN 206:2014, Annex B, Table B1, states that individual test result f_{ci} must be equal or greater than $f_{ck} – 4$, therefore:

- Specimen 1 - 85.56 MPa ≥ 63MPa
- Specimen 2 - 64.40 MPa ≥ 63 MPa
- Specimen 3 - 80.80 MPa ≥ 63 MPa.

And mean value (f_{cm}) must be equal or greater than f_{ck} + 1

- Mean value 76.92 MPa ≥ 68 MPa.

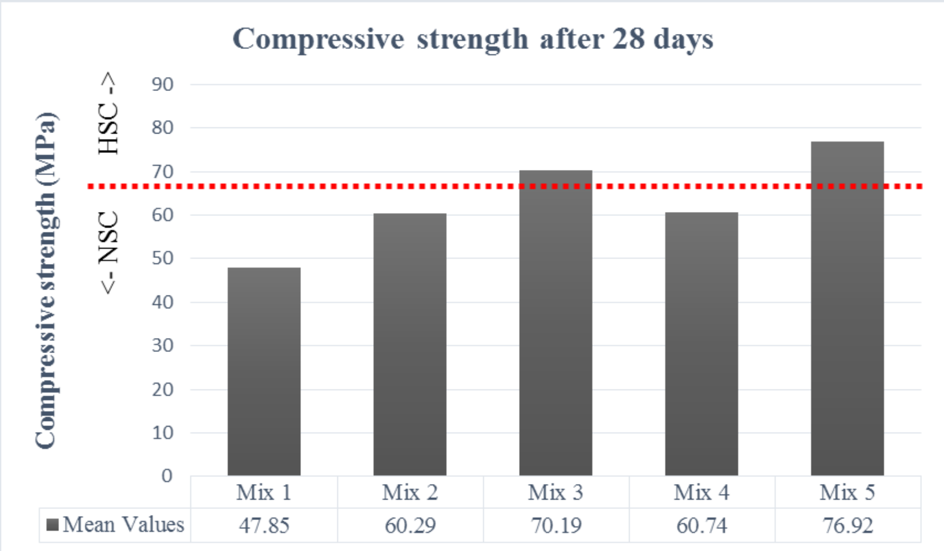


Figure 41 – Graphical Presentation of Results

CONCLUSION

After research exposed in this study, that also included experimental part, the following conclusions can be expressed:

- Concrete technology in Bosnia and Herzegovina is backwards when compared to the world's concrete technology. Routinely accomplished concrete structures in the rest of the world are still in laboratories or don't exist at all in Bosnia and Herzegovina;

- Experimental study conducted in this research resulted in confirmation that there are indeed adequate conditions and domestic constituents in Bosnia and Herzegovina able to produce high–strength concrete and there is a possibility for high–strength concrete production in Bosnia and Herzegovina;

- This study explored the influence of superplasticizer and silica fume on hardened concrete properties–referring to characteristic compressive strength. For experimental purposes of this study, five different concrete mixtures were designed and each was produced in three cube specimens, with dimensions of 15 x 15 x 15 cm. Aggregate composition and content of cement remained equal for every single mixture, while variations were delivered through addition of superplasticizer and different contents of silica fume, with a decrease of water content and with that, a decrease of W/C ratio. Process of production, curing, and testing was done in accordance to BAS EN 206: 2014 and BAS EN 12390 Parts 1, 2, 3 and 4 at the Institute for Material Testing at the Faculty of Technical Sciences, University of Bihać. Use of superplasticizer reduced the W/C ratio from 0.52 to 0.35 and showed noticeable rise in concrete compressive strength after 28 days. Use of superplasticizer and decreased W/C ratio remained for the next two mixtures; addition of silica fume in 5 percent of total cementitious content was added to increase compressive strength of concrete Mix Design 3. Thus, such mixture resulted with the first designated class of high–strength concrete according to BAS EN 206. However, ten percent of silica fume did not gradually continue with the increase in compressive strength but rather approximately repeated the results of concrete Mix Design 2, only using a superplasticizer. Last concrete mixture in this study contained 15 percent of silica fume and showed the highest compressive strength when compared to the rest of the mixtures, as expected; criteria in BAS EN 206: 2014, Annex B, Table B1, classified this mix as, the first designated class of high–strength concrete C 55/67;

- By implementing background theory from qualitative researches and standards into the experimental analyses, this study provided all arguments which could accelerate and increase development of concrete technology in Bosnia and Herzegovina, and at the same time enhance the high quality of domestic constituent materials necessary for concrete technology development;

- Further research needs to be focused to mixing and proportioning of the various concrete mixtures, with more mix tests and continuing testing of properties through concrete specimens for high-strength concrete;

- Through experimental analyses, this research study proved that there is an ability and opportunity for high–strength concrete production in Bosnia and Herzegovina. As long as concrete is described as the mixture of raw materials, and artificially produced new material, there are numerous “what ifs” to be consider and to questions of what would give the best possible results. Therefore, future work may refer to types and locality of raw material by focusing on aggregate type, or its chemical, physical and mechanical properties. Cement, water, silica fume and superplasticizer are already those of the best quality in the area of Bosnia and Herzegovina. Thus, in the future work, aggregate types included in high–strength concrete mix designs may be chosen from other parts of Bosnia and Herzegovina;

- Also, future work may rely on economic analyses by comparing the use of conventional normal strength concrete and high–strength concrete, amount of raw material used, mixed materials and costs through specific equal study cases, with the same architectural and structural designs, but with different structural materials.

REFERENCES

[1] Abrams D. A, 1920, “Design of Concrete Mixtures”, Structural Materials Research Laboratory, Lewis Institute, Chicago, USA;

[2] Annadurai A, Ravichandran A, 2014, "Development of Mix Design for High Strength Concrete with Admixtures", IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 10, Issue 5, pp 22 - 27;

[3] Bajramović E, Džidić S, 2017. “Accredited Laboratories in Process of Introducing Construction Products to EU Market”, Book of Proceedings of the 3rd International Scientific and Professional Conference “SFERA 2017- Concrete Technologies”, organized by SFERA doo Mostar, Faculty of Civil Engineering of the University “Džemal Bijedić” Mostar, Faculty of Civil Engineering of the University of Mostar, Faculty of Architecture and Civil Engineering of the University of Niš and Faculty or Technology of the University of Tuzla, March 23-24, 2017, Mostar, Bosnia and Herzegovina, pp 100 - 105;

[4] Caldarone M. A, 2009. “High-Strength Concrete - A Practical Guide”, Taylor & Francis Group, London and New York, UK and USA, ISBN10: 0-415-40432 (hbk), ISBN 13: 978-0-415-40432-7 (hbk);

[5] Daoud O. M. A, Sayacly H. S, 2013. “Production and Properties of High - Strength Concrete for Heightening Concrete Dam in Sudan.”, Intt. J. of GEOMATE, June, 2013, Vol. 4, No. 2 (Sl. No. 8), Geotec., Const. Mat. & Env. ISSN: 2186-2982(P), 2186-2990(O), Japan, pp. 539 - 545;

[6] Džidić S, 2002. “Residual Pressure Bearing Capacity of the Concrete and HSC Affected by High Temperatures”, IV International Scientific Research Symposium “Metallic and Nonmetallic Materials – Production – Properties – Application”, Proceedings, Zenica, Bosnia and Herzegovina, pp 203 - 209;

[7] Džidić S, 2015. “Otpornost betonskih konstrukcija na požar”, International BURCH University Sarajevo, Bosnia and Herzegovina, ISBN 978-9958-834-47-9; COBISS.BH-ID 22444550;

[8] Džidić S, Kozlica S, Ćehić E, 2016. “Experiences in Testing of Compressive Strength of Concrete during Construction of Waste Water Collection and Treatment Plant in Bihać according to BAS EN 206:2014”, Book of Proceedings of the 12th International Scientific and Professional Conference on Contemporary and Theory and Practice in Construction, University of

Banja Luka Faculty of Architecture, Civil Engineering and Geodesy, December 7-8, 2016, Banja Luka, Bosnia and Herzegovina, ISBN 978-99976-663-3-8, UDK 69:71/72(082)(0.034.2) 624(082)(0.034.2), COBISS.RS-ID 6340024, pp 495 - 502;

[9] Džidić S, Okugić H, Bajramović E, 2017, "Studije o betonu 2016-2017; International BURCH University Sarajevo, Bosnia and Herzegovina, CIP 624.012.4, ISBN 978-9958-834-57-8, COBISS.BH-ID 24495622 ;

[10] Eilzer W, Stockmann R, Weigel F, 2003. "Planung und Ausführung der Muldebrücke Glauchau in Hochleistungsbeton B85", 13. Dresdner Brückenbausymposium. Dresden, Germany, pp 139 – 154;

[11] Felić A, Haračić E, 2015. "Studija ekonomske opravdanosti eksploatacije dolomita "Džehveruša" Cazin", EDA, Rad Putevi, Bihać, Bosna i Hercegovina;

[12] Kovačević I, Džidić S, 2018, "High-Rise Buildings–Structures and Materials", International BURCH University Sarajevo, March 2018, Sarajevo, Bosnia and Herzegovina, ISBN I978-9958-834-58-5; COBISS.BH-ID COBISS.BH-ID 24904454;

[13] McCormac J. C, Brown R.H, 2014. "Design of Reinforced Concrete", 9th Edition, ACI 318-11 Code Edition, John Wiley & Sons, Inc. USA, SBN: 978-1-118-12984-5, ISBN: 978-1-118-43081-1 (BRV);

[14] Mosley B, Bungey J, Hulse R, 2007. "Reinforced Concrete to Eurocode 2", Houndmills, Basingstoke, Hampshire Palgrave MacMillan, RG 21 6XS, UK and New York, USA ISBN-13 978-0-230-50071-6, ISBN-10 0-230-50071-4;

[15] Nepomuceno M. C. S, Oliveira L. A. P, Lopes S. M. R, 2014."Methodology for the Mix Design of Self-Compacting Concrete, Using Different Mineral Additions in Binary Blends of Powders", Construction and Building Materials, 64:82–94, DOI 10.1016/j.conbuildmat.2014.04.021;

[16] Portland Cement Association, 1994. "High-Strength Concrete", Concrete Technology Today, Volume 15, Number 1, March 1994, 5420 Old Orchard Road, Skokie, Illinois 60077-1083, USA;

[17] Ramesh D, Murali S, Balaji S, Ganesan V. 2015. "Design of High-Strength Concrete Mix 60 and Investigation of its Strength", International Journal of Innovative Research in Science, Engineering and Technology, Vol. 4, Issue 10, October 2015, ISSN (Online): 2319-8753, ISSN (Print): 2347-6710, pp 10157 -10161,

[18] Rashid M. A, Mansur, M. A, 2009. “Considerations in Producing High-Strength Concrete.” Journal of Civil Engineering (IEB), 37(1) (2009), pp 53 - 63;

[19] Rogić V, Šaravanja K, Dasović D, 1986. “Tehnologija dobivanja i istraživanja betona visokih marki sa amfornom mikrosilikom i superplastifikatorima - beton sa mikrosilicom (BSM).”, XIII Kongres Jugoslavenskog društva za visoke brane. Mostar, Univerzitet Džemal Bjedić u Mostaru, Zbornik radova Mostar, Jugoslavija, pp 43 – 65;

[20] Sivakumar N, Muthukumar S, Sivakumar V, Gowthaw D, Muthuraj V, 2014. “Experimental Studies on High-Strength Concrete by Using Recycled Coarse Aggregate”, International Journal of Engineering And Science Vol. 4, Issue 01 (January 2014), ISSN(e): 2278-4721, ISSN (p):2319-6483, pp 27 - 36;

[21] Skazlić M, Tomičić I, 2006. “Arimiranobetonski elementi od betona visokih čvrstoća naprezani savijanjem”, GRAĐEVINAR 58 (2006) 8, Journal of the Croatian Association of Civil Engineers, Zagreb, Croatia, ISSN 1333-9095, pp 631 - 640;

[22] Šahinagić-Isović, M, Markovski G, Ćećez M, 2012. “Shrinkage Strain of Concrete - Causes and Types”, GRAĐEVINAR, 64 (2012) 9, Journal of the Croatian Association of Civil Engineers, Zagreb, Croatia, ISSN 1333-9095, pp 727 - 734;

[23] Tuppinger H, Amlacher H, 2002. “Hochleistungsbeton im Brückenbau”, Montanuniversität Leoben, Lehrstuhl für Gesteinshüttenkunde, Lieserbrücke, 9851, Austria;

[24] Umar H. M, Naseer S, Sohail M. R, Hussain M, 2014, “Experimental Study on Strength of Concrete using Silica Fumes as Supplementary Cementitious Material“, First International Conference on Emerging Trends in Engineering, Management and Sciences, December 29-30, 2014 Peshawar, Pakistan;

[25] Vihutha D, Velu S, Tamilnathan R, Vijay S, Vigneshwaren A, 2016. “Aggregate on Self Compacting Concrete Using M 70 Grade.” International Journal for Research in Applied Science and Engineering Technology (IJRASET), Volume 4, Issue IV, April 2016, ISSN: 2321-9653, pp 215 – 230;

[26] Wu D, Sofi M, Mendis P, 2010. “High-Strength Concrete for Sustainable Construction”, Proceedings of the International Conference on Sustainable Built Environment, (ICSBE-2010), University of Moratuwa, Society for the Promotion of Science, Kandy, Japan, pp 434 – 442.

STANDARDS

[27] BAS EN 12350-2:2010, Testing fresh concrete – Part 2: Slump – test, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[28] BAS EN 197-1:2013, Cement - Part 1: Composition, specifications and conformity criteria for common cements, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[29] BAS EN 12390-1:2013, Testing hardened concrete - Part 1: Shape, dimensions and other requirements for specimens and moulds, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[30] BAS EN 12390-2:2010, Testing hardened concrete - Part 2: Making and curing specimens for strength tests, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[31] BAS EN 12390-3:2010, Testing hardened concrete - Part 3: Compressive strength of test specimens, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[32] BAS EN 12390-4:2003, Testing hardened concrete - Part 4: Compressive strength - Specification for testing machines, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[33] BAS EN 206:2014 Concrete - Specification, performance, production and conformity, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[34] BAS EN 933 Tests for geometrical properties of aggregates, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[35] BAS EN 934-2+A1:2013 Admixtures for concrete, mortar and grout - Part 2: Concrete admixtures - Definitions, requirements, conformity, marking and labelling, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina.

INTERNET RESOURCES

[36] http://designarchitectureart.com/2016/12/avoid-segregation-concrete/

[37] http://file.scirp.org/Html/7-7700639_19836.htm

[38] http://skyscraperpage.com/diagrams/?searchID=79170701

[39] http://www.betoni.hr/Portals/2/Docs/Kontrola%20proizvodnje%20betona.pdf?timestamp=1448456081168

[40] http://www.bsibh.com/en/products.htm

[41] http://www.civilengineeringforum.me/supplementary-cementing-materials/

[42] http://www.engineeringintro.com/concrete/admixtures/admixtures-types-mineral-and-chemical-admixtures/

[43] http://www.kakanjcement.ba/

[44] http://www.katastar.ba/geoportal/preglednik/

[45] http://www.kakanjcement.ba/bs/cement

[46] http://www.sciencedirect.com/science/article/pii/S0950061817302702

[47] https://egy.sika.com/dms/getdocument.get/b4245150-*e1f5.../Plastocrete®%20-N.pdf*

[48] https://structurae.net/structures/iwahana-rail-overpass

[49] https://www.iha.com/Short-term-rentals-Rivedoux-plage/!q/

[50] https://www.linkedin.com/pulse/detailed-methods-slump-test-check-consistency-fresh-concrete-g-k/

[51] https://www.nb.no/ekofisk/funn_eng.pdf

[52] https://www.nrc-cnrc.gc.ca/ctu-sc/ctu_sc_n31

[53] https://www.northernarchitecture.us/american-concrete/modulus-of-elasticity.html

[54] https://www.slideshare.net/mmrabir/investigation-of-compressive-strength-of-concrete-made-by-different-types-of-aggregate-in-marine-environment

STUDY 3

CONCRETE QUALITY CONTROL ACCORDING TO EUROPEAN STANDARDS

CASE STUDY

CONSTRUCTION OF THE WASTE WATER TREATMENT PLANT IN BIHAĆ

Sabina Kozlica

Sanin Džidić

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION TO WASTE WATER TREATMENT PLANT CONSTRUCTION PROJECT IN BIHAĆ

Construction of Waste Water Treatment Plant (WWTP) was one of two components of the overall project – Collection and Treatment of Waste Water in Bihać.

With the City of Bihać as the Employer[5], and KfW Bank as the funder, partners in this project also included European Union, the Government of Federation of Bosnia and Herzegovina, Environmental Fund of the Federation of Bosnia and Herzegovina and Agency for Water Resources of the Sava River Region.

The contract between the Employer and the Contractor was modeled on FIDIC[1] type of contract, outlined by the Yellow Book, and had a total value of about 10,800,000 €, including all parts of the project (civil, mechanical, electrical, and overall process). Since the contract was based on the FIDIC Yellow Book, the Contractor performed both the design and construction works.

The importance of the Project lies in the improvement of the health and environmental conditions of the local population, and, in a broader sense, health and environment of the region: River Una flows into River Sava, which than flows into Danube, the second longest and the second most voluminous river in Europe (the first being River Volga), and the longest river in the European Union. Project documentation predicted construction of the facilities in two phases: Phase I - for loads up to the year 2030, and the Phase II - for the load after the year 2030. Scope of the project was to design and construct facilities for Phase I, designed for capacity of 55,000 PE, while extension capacities for the facilities of Phase II (capacity 82,500 PE) had been taken into consideration along with the design of the first phase.

Civil works on construction of the WWTP started with the construction of access road, which was performed in two phases, with sections 0 + 0,000 m – 0 + 687,552 m in the first phase, and sections 0 + 687,552 m – 1+245,872 m in the second phase.

Since the land where the construction of the WWTP was planned had been characterized by poor soil quality in terms of its bearing capacity, the design considered the installation of FDP piles under facilities that would be part of the plant. Namely, a total of 681 piles, 61 cm in diameter, 3.50 m axial distance, with an average depth of 17-19 m, were installed below the facilities of the plant. The technology used to make these piles was one that was seldom used in this region and represented one of many novelties in construction. Such installation technology permits quick mounting of piles by pushing the piles into muddy soil, then filling the boreholes with

[5] International Federation of Consulting Engineers (acronym for its French name Fédération Internationale Des Ingénieurs-Conseils)

fluid-in-consistency fresh concrete, and finally positioning reinforcement bars in fresh concrete. It is important to note that this installation technology allows considerable time savings, as we can see from the fact that all piles were built in about 60 days, with an average installation of about 20 piles per day.

In the construction sense, construction of buildings is a common undertaking, both in terms of architecture of buildings and in terms of materials used during construction. All facilities were simultaneously constructed according to the construction schedule and actual weather conditions. Since 22 out of 25 facilities of the first phase of the project were reinforced concrete structures, a lot of attention was dedicated to quality control of concrete.

Figure 1 - WWTP in Construction

It is quite significant to mention that the entire complex of the WWTP is secured by the flood protection dam; it is designed to withstand the impact of flood and earthquake. Flood protection dam is about 700 m long and divided into 35 sections. The crown of the dam is 3 m wide, while the bottom of the dam at its widest section is about 17 m wide. Total height of the dam is 4 m. The dam was done with the mixture of soil and sand, at the ratio 80% soil to 20% sand. Each layer of the dam was compressed to compressibility modulus of 25 MPa. The clay core is placed inside the dam, 2 m below the subbase, and extends to the top of the dam. Riprap layer, which prevents the body of the dam from being damaged by flooding water is placed on the outer side of the dam.

Figure 2 – Finalized WWTP

The treatment process is conducted through an extended aeration technology. Even though it presented a bit more expensive solution than suggested SBR (Sequencing Batch Reactor) technology, this type of technology, chosen by the main criterion– quality of the treated waste water, was much more reliable in the sense it lowered the risk of work termination in case of any damage (much lower expenses).

Waste water treatment plant is a group of facilities that participate in the process of treatment of domestic and industrial waste water. It consists of 25 facilities that were constructed in the first phase of the project. In the case of population growth, expansion of capacities of some facilities is ensured.

Facilities of the WWTP and their function are presented as follows:

- Inlet Structure – the role of this facility is to remove large waste which is collected within a 5 m^3 volume container. This object measures 1475 cm x 526 cm, and 760 cm in height. This facility is equipped with coarse screens that hold coarse materials, which are then separated into containers, while the waste water is carried further into the following facility by four pumps;

- Fine Screens Building – waste water that flows into from the previous facility flows through fine screens that can be find in this facility. These screens are supposed to hold smaller materials that pass through the coarse screens, such as plastic bags, fiber and similar materials. This building measures 975 cm x 1250 cm, and 1326 cm in height;

- Aerated Grit and Grease Chamber – this facility has a function of grit and grease removal. Waste water that flows in from Fine Screens Building, free of solid waste, now undergoes through a process of removal of grit and grease by using blowers;

- Activated Sludge Tank – this facility has a role of biochemical treatment of waste water. It is physically divided into two equal sets of chambers which are capable of working independently. Both sets of chambers are divided into three zones. Waste water flows in from the Aerated Grit and Grease Chambers into the first zone - anoxic zone, where conditions for nitrogen and phosphorus removal are established. Water then flows into anaerobic zone, where processes of denitrification and dephosphorisation occur. From this zone, water flows into aerobic zone, the biggest zone among the three, where biological process of removal of leftover microorganisms carries on. Water flows through overflow wall to Distribution Chamber to Final Sedimentation Tanks, where, in case that previous processes haven't removed pollutants satisfyingly, the additional chemical purification takes place. This facility is the biggest facility of the WWTP, with the dimensions 9765 cm x 4120 cm x 690 cm;

- Final Sedimentation Tanks – these facilities receive biochemically treated water, and sedimentation process takes place here. As rotating scrapers rotate around the diameter of these facilities, larger particles in water tend to sediment out, while pure water tends to stay on the surface (supernatant). Supernatant flows further into the UV Disinfection Channel, while deposited sludge is transported into two directions: one part goes into the anoxic zone, while the other part goes into the Excess Sludge Thickener. These facilities are circular in shape with diameter of 2880 cm and 804 cm in height, measured from the bottom of the base slab;

- UV Disinfection Channel – this facility belongs to the optional treatment section. It is quite narrow and long facility equipped with UV lamps which kill DNA of microorganisms (if there are left any) and in that way prevent their reproduction in water. Purified water flows out of these facilities into the recipient – River Una. Dimension of this facility is 3775 cm x 175 cm x 5,05 cm at the deepest point measured from the bottom of the base slab;

- Return Excess Sludge Pumping Station – this facility contains pumps that enable movement of return sludge to the anoxic zone and enable excess sludge to be transported to the Excess Sludge Thickener. The size of this facility is 735 cm x 820 cm x 620 cm at the deepest point measured from the bottom of the base slab;

- Excess Sludge Thickener – this facility is circular in shape and functions pretty similar to the Final Sedimentation Tanks. Rotating scraper enables the excess sludge to be deposited at the conical bottom of the facility, while free water is separated at the surface. Deposited sludge is then further transported into a Dewatering Building, while separated water flows by Filtrate and Supernatant pipeline into the Inlet Structure, and to the very beginning of the treatment process. Size of this facility is 1380 cm x 770 cm measured from the deepest point and bottom of the base slab;

- Dewatering Building – this facility contains two belt presses that compress and dry thickened sludge that comes from Excess Sludge Thickener, and transport it to the Sludge Storage Area through a conveyor system. This is one of the three masonry facilities among 22 entirely reinforced concrete structures. This facility, rectangular in shape, measures 2430 cm x 1230 cm x 600 cm;

- Sludge Storage Area – this facility, with its size of 3000 cm x 1880 cm x 315 cm measured from the deepest point and bottom of the base slab, provides the space for thickened and dried sludge, capable to store sludge for about 60 days; after that, the sludge needs to be transported further to another depot. This is the point where the main treatment cycle ends;

- Transformer and Blower Station – this facility is masonry structure, 1830 cm x 1230 cm x 492 cm big. It enables the work of membrane diffusors in the Activated Sludge Tank by blowing hot air into them which is produced by 4 working blowers. A generator is also placed in this facility, in case of an electricity cutoff occurrence;

- Administration Building – all offices, laboratory and SCADA system are placed in this building. This is the monitoring place where all actions on the WWTP are controlled. This facility measures 2230 cm x 1230 cm x 501,5 cm;

- Odor Control Unit – this facility is one of the smallest facilities of the WWTP that has a purpose of collecting unpleasant odors from the environment, as well as from the Dewatering Building through the appropriate air pipeline. This process occurs through biomass which is placed on the top of the unit;

- $FeCl_3$ Dozing Unit – Ferric Chloride Dozing Unit is a small facility that holds a tall container of $FeCl_3$ that is used in the process of chemical treatment of water in Activated Sludge Tank;

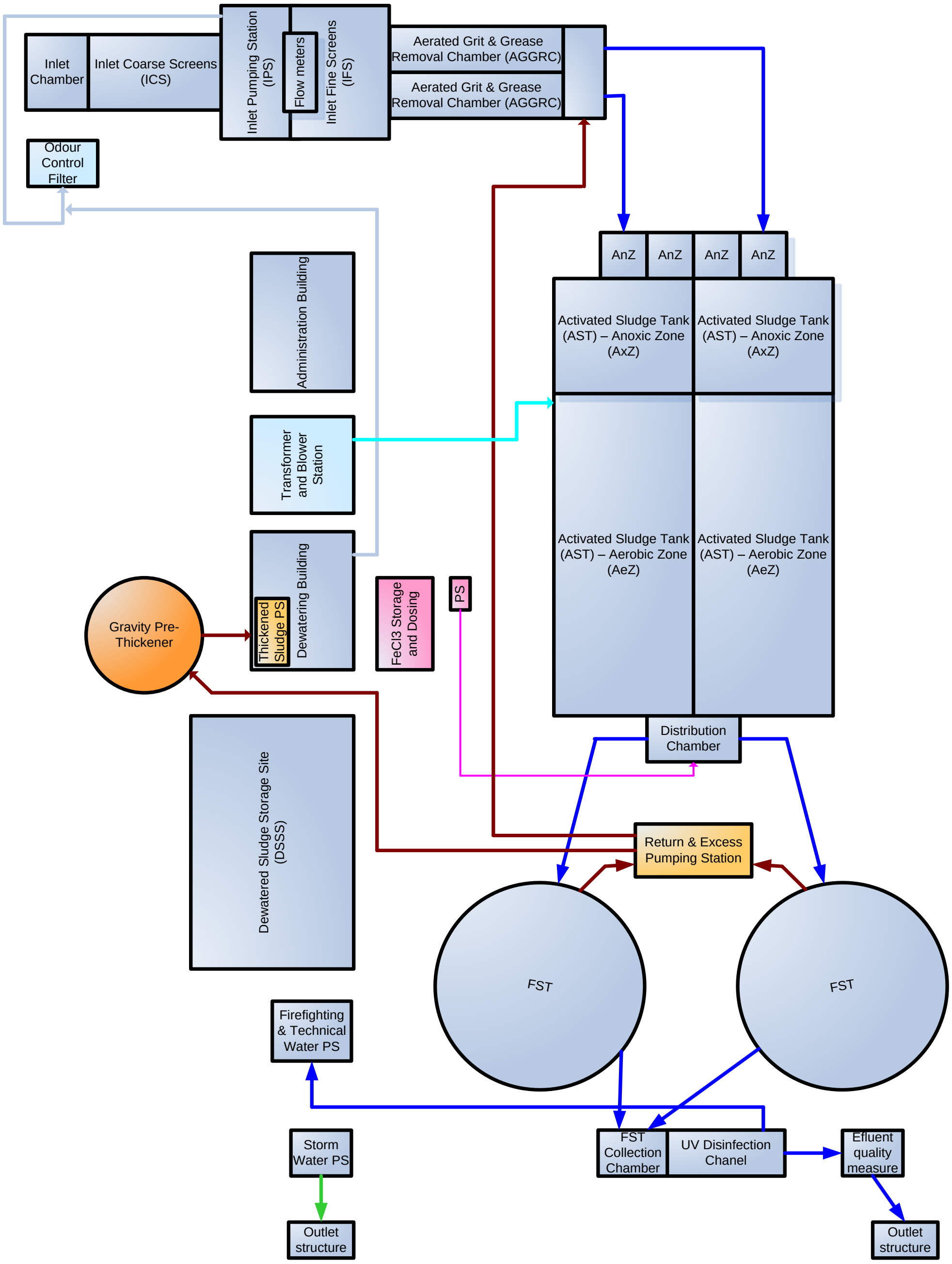


Figure 3 – Scheme of Treatment Process [11]

- Effluent Tank and Pumping Station for Firefighting and Technical Water – this facility stores storm water that drains throughout the WWTP and contains pumps that pump water to hydrants all over the WWTP. This water is used for cleaning, washing, and overall maintenance purposes. The dimension of this facility is 1065 cm x 460 cm x 725 cm measured from the deepest point and bottom of the base slab;

- Storm Water Pumping Station – this facility has two pumps that are immersed in drained water which is collected from the WWTP site through the Internal Site Drainage pipeline. This water is then transported by the pipeline to the Effluent Tank and Firefighting Pumping Station. Rectangular in shape, it has following dimensions: 230 cm x 230 cm x 525 cm.

QUALITY ASSURANCE PROGRAM

Quality Control Program defines a set of quality measures to be taken in order to accomplish quality requirements defined by the project. The complexity of the Quality Assurance (QA) program is defined depending on the complexity of the project itself.

Since the construction of the WWTP consisted of three main parts, the QA program was branched into three main directions – electrical, mechanical and civil engineering part, which was the focus of this study.

Quality control program was present during all phases of construction of the WWTP, starting from conceptual design and investment study, through the design, and to the actual construction and maintenance. On site implementation of quality control of concrete was conducted through the quality plan for construction, site laboratory and records of quality control procedures. Site documentation regarding quality control program included:

- Specification of materials and construction products;
- Quality plan for construction of concrete structures;
- Laws, regulations, technical regulations, standards;
- Records of the control activities conducted in situ;
- Evidences about the quality of built-in concrete (in situ test reports conducted); and
- Other documents, licenses, and permits actually present on site.

There are several sources that QA program is based on. Namely, these sources include FIDIC regulations, BAS EN 206:2014 requirements, tender requirements and national regulations and requirements on construction.

Founded in 1913, FIDIC is charged with promoting and implementing the consulting engineering industry's strategic goals on behalf of its Member Associations and to disseminate information and resources of interest to its members. Today, FIDIC membership covers 104 countries of the world. [44]

FIDIC – Yellow Book, one out of several books published by FIDIC, gave the rough base for making the QA program. In fact, FIDIC represents the main directions in making the international construction contract agreements.

The Yellow Book provides conditions of contract for construction works where the design is carried out by the Contractor. The Yellow Book is therefore applicable to the provision of electrical and/or mechanical plant, and for the design and construction of building or engineering works. Under the usual arrangements for this type of contract, the Contractor designs and provides the works in accordance with the Employer's requirements which may include any combination of civil, mechanical, electrical and/or construction works. [45]

To conclude, every obligation specifically regarding civil works, is derived from the contractual requirements, which lead to the tender requirements.

Derived from the FIDIC regulations, requirements defined by the tender represented specifically defined points that described the Employer´s requirements. This document was the exact foundation for the design of the QA program, since it contained strictly defined and described terms which the Contractor had to fulfill.

In the Employer´s requirements, it could be considered that quality control program regarding reinforced concrete structures could be divided in two segments:

- Quality control of reinforced concrete as a material; and
- Quality control of construction.

Quality control of reinforced concrete as a material also included testing of component materials that concrete consisted of. Since concrete is a composite material, the individual testing of component materials (aggregate, cement and water) as well as testing of concrete material as a whole give the appropriate quality evaluation of the material.

Besides the criteria defined by European and national standards, some of the terms stated in the Employer´s requirements regarding the quality control of reinforced concrete and construction were following:

- The essential step of quality control was to keep accurate and up-to-date records of every concrete casting, where had to inevitably included the following information:
    - ✓ Date, time, weather conditions and temperature;
    - ✓ Results of all concrete tests, including identification;
    - ✓ Number of batches produced, weight and type of cement used, volume of concrete placed, number of batches wasted or rejected;
    - ✓ Concrete class, volume of concrete placed and number of batches used for each location. [8]

- The approval for concrete from a single supplier of ready-mixed concrete will not be given until the Employer is satisfied that the organization and control of the manufacture and delivery of all ready-mixed concrete is in accordance with the Employer's Requirements;
- The type of cement used in the works shall be Portland cement from a single approved source conforming to the requirements of Portland cement in accordance to BAS EN 196;
- The Contractor had to submit to the Employer, free of charge, test certificates relating to each consignment of cement. Each certificate shall show that a sample of the consignment has been tested by the manufacturer or by an approved laboratory and that it complies in all respects with the Employer's Requirements. [8]
- The water used in concrete mixture should comply with BAS EN 1008:2002, obtained from approved source of quality that will not affect the setting time, strength, durability of the concrete or mortar, or the appearance of hardened concrete or mortar by discoloration or efflorescence, or the reinforcement at any age of the concrete or mortar;
- The aggregate had to conform to BAS EN 12620+A1:2009, obtained from natural source, strong and durable, not containing harmful material of sufficient quantity to affect adversely the strength or durability of the concrete or, in the case of reinforced concrete, to attack the reinforcement;
- Fine and coarse aggregates in concrete elements exposed to wastewater had to be equivalent to the high sulphate resistance of the cement (siliceous sand and gravel);
- When consider testing frequency, all common in situ tests, especially slump tests, had to be carried out on samples taken from every transmixer. Precisely, test samples had to be taken at a minimum rate of one set of 6 samples for every 50 m^3 of concrete if the cast quantity is more than $50m^3$. If the cast quantity is less than 50 m^3 per one day, a minimum of one set of 6 cubes will be taken. Generally, test cubes were taken as directed by the supervising engineer. All tasting cubes needed to be marked at the time of casting, with the date, class of concrete and other necessary markings to identify the part of the works from which they are taken; [8] and
- All other terms derived from European and national standards.

Tools for providing the measures of quality control may be separated in several forms:

- Method statements for construction works;
- Requests for approval of material to be used; and
- QA/QC monthly reports.

Method statements represent documentation on the construction process of a specific facility or part of the facility. They contain a description of every step of the construction process, with defined materials that are planned to be embedded, the rate of fresh concrete delivery, any special works, quantities of materials to be used, machinery necessary for construction works. Method statements were written individually for each facility of the WWTP.

Requests for approval of material to be used are special forms, agreed upon between the contractor and oversight engineer, which contain necessary information for a specific material which is the subject of approval. By submitting the Request for approval of material to the oversight engineer, the Contractor provides information on quality of the material which is to be used. This information for the WWTP included following:

- The name of the material/product;
- Short description of the material/product (with scheme or picture available);
- Facility where the material/product will be used;
- Standards and norms related to the material/product;
- Information about manufacturer (address and contact of headquarters);
- Information about supplier (address and contact of headquarters);
- Enclosed documents regarding conformity certificate, laboratory testing report, material/product catalogue, available drawings and schemes; and
- Notes.

The QA/QC monthly report is a document that, in fact, unifies all records for any construction work. Specifically, considering concrete works, this report included all on site filled forms, kept by the contractor and approved by the oversight Engineer. The forms regarding concrete works included:

- Drilling logs for piles;
- Inspection logs for reinforcement and formwork;
- Inspection logs for quality and delivery control of concrete before casting;

- Test reports submitted by the authorized laboratory and Contractor conducted on site for the current month.

REQUEST FOR APPROVAL OF MATERIAL(S)

Request No.: 63	Submittal No.: 1

Date:	Time:
Submitted by:	Signature:

Description of Material(s): (Including size, class, type, strength, accessories etc.)

Reference to Technical Specifications:

Clause(s):

Applicable Standard(s):
Attachment:

Reference to Contract Drawing(s):

Drawing No.(s):

Name of Manufacturer:

Postal Address:
Phone:
Fax:

Attached Documentation: (Tick the appropriate)

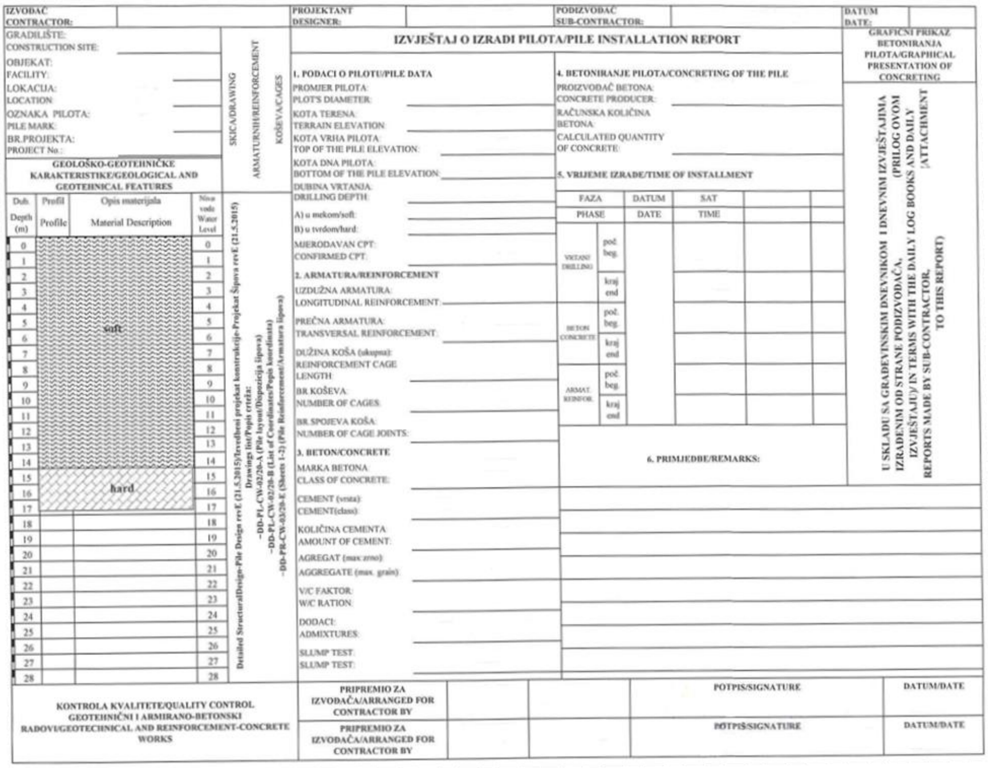
O Manufacturer's Documentation	O Test Reports
O Certificates	O Material Sample(s)

The material(s) is/are: (Tick the appropriate)

O Approved (In compliance with Contract Documents)
O Not Approved (Not in compliance with Contract Documents)
O Reason for Disapproval:

Figure 4 – Request for Approval of Material Form [14]

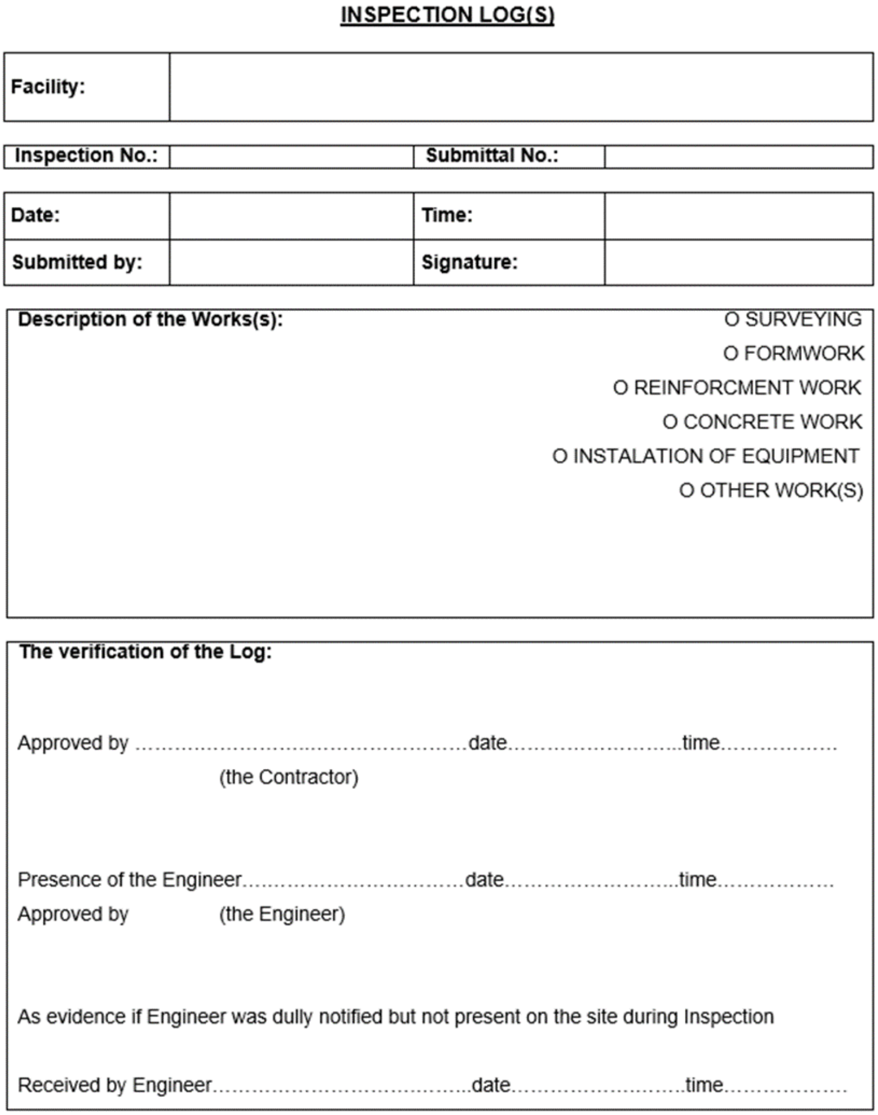
Drilling log for piles is actually a technical sheet for every single pile. In addition to the geological information of pile drilling, it contains information about quantity of concrete placed in the pile installation, as well as concrete consistency testing results, which will be discussed in detail in the forthcoming chapters of this study.



IZVOĐAČ
CONTRACTOR:
PROJEKTANT
DESIGNER:
PODIZVOĐAČ
SUB-CONTRACTOR:
DATUM
DATE:
GRADILIŠTE:
CONSTRUCTION SITE:
OBJEKAT:
FACILITY:
LOKACIJA:
LOCATION:
OZNAKA PILOTA:
PILE MARK:
BR.PROJEKTA:
PROJECT No.:
GEOLOŠKO-GEOTEHNIČKE KARAKTERISTIKE/GEOLOGICAL AND GEOTECHNICAL FEATURES
SKICA/DRAWING
ARMATURNI/REINFORCEMENT
KOŠEVI/CAGES
IZVJEŠTAJ O IZRADI PILOTA/PILE INSTALLATION REPORT
GRAFIČNI PRIKAZ BETONIRANJA PILOTA/GRAPHICAL PRESENTATION OF CONCRETING
1. PODACI O PILOTU/PILE DATA
PROMJER PILOTA:
PILOT'S DIAMETER:
KOTA TERENA:
TERRAIN ELEVATION:
KOTA VRHA PILOTA:
TOP OF THE PILE ELEVATION:
KOTA DNA PILOTA:
BOTTOM OF THE PILE ELEVATION:
DUBINA VRTANJA:
DRILLING DEPTH:
A) u mekom/soft:
B) u tvrdom/hard:
MJERODAVAN CPT:
CONFIRMED CPT:
2. ARMATURA/REINFORCEMENT
UZDUŽNA ARMATURA:
LONGITUDINAL REINFORCEMENT:
PREČNA ARMATURA:
TRANSVERSAL REINFORCEMENT:
DUŽINA KOŠA (ukupna):
REINFORCEMENT CAGE LENGTH:
BR KOŠEVA:
NUMBER OF CAGES:
BR SPOJEVA KOŠA:
NUMBER OF CAGE JOINTS:
3. BETON/CONCRETE
MARKA BETONA:
CLASS OF CONCRETE:
CEMENT (vrsta):
CEMENT(class):
KOLIČINA CEMENTA:
AMOUNT OF CEMENT:
AGREGAT (max zrno):
AGGREGATE (max. grain):
V/C FAKTOR:
W/C RATION:
DODACI:
ADMIXTURES:
SLUMP TEST:
SLUMP TEST:
4. BETONIRANJE PILOTA/CONCRETING OF THE PILE
PROIZVOĐAČ BETONA:
CONCRETE PRODUCER:
RAČUNSKA KOLIČINA BETONA:
CALCULATED QUANTITY OF CONCRETE:
5. VRIJEME IZRADE/TIME OF INSTALLMENT
FAZA | DATUM | SAT
PHASE | DATE | TIME
VRTANJE DRILLING: poč. beg. / kraj end
BETON CONCRETE: poč. beg. / kraj end
ARMAT. REINFOR.: poč. beg. / kraj end
6. PRIMJEDBE/REMARKS:
U SKLADU SA GRAĐEVINSKIM DNEVNIKOM I DNEVNIM IZVJEŠTAJIMA IZRAĐENIM OD STRANE PODIZVOĐAČA, IZVJEŠTAJU) IN TERMS WITH THE DAILY LOG BOOKS AND DAILY REPORTS MADE BY SUB-CONTRACTOR. (PRILOG OVOM ATTACHMENT TO THIS REPORT)
Dub. Depth (m) | Profil Profile | Opis materijala Material Description | Nivo vode Water Level
0–28
soft
hard
Detailed Structural Design-Pile Design revE (21.5.2015)/Izvedbeni projekat konstrukcije-Projekat Šipova revE (21.5.2015)
Drawings list/Popis crteža:
-DD-PL-CW-02/20-A (Pile layout/Dispozicija šipova)
-DD-PL-CW-02/20-B (List of Coordinates/Popis koordinata)
-DD-PR-CW-02/20-E (Sheets 1-2) (Pile Reinforcement/Armatura šipova)
KONTROLA KVALITETE/QUALITY CONTROL GEOTEHNIČNI I ARMIRANO-BETONSKI RADOVI/GEOTECHNICAL AND REINFORCEMENT-CONCRETE WORKS
PRIPREMIO ZA IZVOĐAČA/ARRANGED FOR CONTRACTOR BY
POTPIS/SIGNATURE
DATUM/DATE
PRIPREMIO ZA IZVOĐAČA/ARRANGED FOR CONTRACTOR BY
POTPIS/SIGNATURE
DATUM/DATE

Figure 5 – Drilling Log for Piles [14]

Inspection logs for reinforcement and formwork were used for inspection of correctness, and were in accordance to the design of installed reinforcement and formwork. These inspections, in fact, represent a form of approval of preparations done for the placing of concrete. Prior to inspection, the contractor is supposed to officially invite the oversight engineer to the inspection and approval of performing concrete works. Before the purchase of concrete from concrete plant, the oversight engineer and contractor representatives conduct the inspection of reinforcement and formwork by controlling and comparing the reinforcement and formwork works to the design specifications. As soon as it is concluded that all elements of the facility planned for concrete works are done according to the approved design specifications, the oversight Engineer provides the approval to the contractor for the concrete works. In addition to these inspections, for the sake of health and safety on construction site, the inspection of dimensions and stability of scaffold is also conducted before concrete works. An example of inspection log for reinforcement and formwork is shown in the following figure.



INSPECTION LOG(S)

Facility:			
Inspection No.:		**Submittal No.:**	
Date:		**Time:**	
Submitted by:		**Signature:**	

Description of the Works(s):

O SURVEYING
O FORMWORK
O REINFORCMENT WORK
O CONCRETE WORK
O INSTALATION OF EQUIPMENT
O OTHER WORK(S)

The verification of the Log:

Approved by ……………………………………………date……………………..time………………
(the Contractor)

Presence of the Engineer……………………………date……………………..time………………
Approved by (the Engineer)

As evidence if Engineer was dully notified but not present on the site during Inspection

Received by Engineer………………………………..date……………………..time………………

Figure 6 – Inspection Log for Reinforcement and Formwork [14]

Inspection logs for quality and delivery control of concrete were mandatory before and during the concrete placing. These inspection logs included all relevant information about concrete properties of every batch of concrete.

The log included the following information:

- Facility that is planned for concrete works and number of inspection;
- Date and time, signature of personnel conducting the control;

- Description of concrete type;
- Batch lot number; and
- Records of in situ testing of fresh concrete (delivery note number, concrete consistency, contained air, outside temperature, concrete temperature, specific weight).

QUALITY AND DELIVERY CONTROL OF CONCRETE BEFORE PLACING
KONTROLA KVALITETA I ISPORUKE BETONA PRIJE UGRADNJE

Facility/Objekat:	

Inspection No./ Pregled br.:	

Date/Datum:		**Time/Vrijeme:**	
Kept record(s) by / Za izvještaj zadužen:		**Signature/Potpis:**	

Description of the Concrete type/Opis tipa betona:	
Class of concrete strength/Razred čvrstoće	
Receptura broj/oznaka - Concrete mix design number/mark	
Cement/Cement	
Water-cement ration / Vodocementni omjer	
Exposure class/Razred izloženosti	
Consistence/Konzistencija	
Maximum grain aggregate/Maksimalno zrno agregata	
Watertightness of concrete/Vodonepropusnost betona	
Additives /Aditivi	
Total amount of concrete/Ukupna količina betona	
Position/Pozicija	
Total number of batch lot/Ukupni broj partija	

Batch lot number/Partija broj:	
Number of samples (concrete cube/roller)/ Broj uzoraka (kocka/valjak)	_____ pcs/batch lot/ kom/partiji
Sampling period/Period uzorkovanja	______2016.
Amount of delivered concrete/Dopremljeno betona	______ m^3
Total amount of delivery notes (For period)/Ukupno otpremnica (za period)	______ pcs (specification in attachment) kom (specifikacija u prilogu)
Amount of placed concrete /Ugrađeno betona	______ m^3

Figure 7 – Inspection Log for Quality and Delivery Control of Concrete before Placing [14]

Test reports submitted by the approved laboratory and done by the contractor in situ for a given month were documents issued by the laboratory that officially kept and represented the testing records according to the contract agreement.

In addition to all aforementioned, it is necessary to highlight some aspects of the BAS EN 13670:2011 standard - Execution of concrete structures. This European standard gives common requirements for execution of concrete structures.

This standard has three major functions:

- To transfer the requirements set during design from the designer to the contractor i.e. to be link between design and execution;
- To provide a set of standardized technical requirements for the execution when ordering a concrete structure; and
- To serve as a check list for the designer to ensure transfer of all relevant technical information for the execution of the structure to contractor. [36]

Also, this standard assumes the following:

- The availability of a comprehensive design of the structure;
- A project management in charge of the supervision of the works which will ensure the execution of a conforming structure; and
- A site management which will take manage the organization of the works and ensure the correct and safe use of the equipment and machinery, the satisfactory quality of materials, the execution of a conforming structure and its safe use up to the delivery of the works. [36]

In terms of construction execution of reinforced concrete structures, this standard covers following aspects:

- Documentation - project specification and execution documentation – which implies quality plan and method statements;
- Falsework and formwork requirements - which are controlled by the means of inspection logs described earlier;
- Reinforcement requirements - which are controlled by the means of inspection logs described earlier;
- Casting of concrete - requirements regarding the process of concrete works: steps for determining the concrete characteristics before the unload, measure of control prior to concrete works (visual examination of the element prepared

for concrete cast), measures during the placing and compaction (proper use of vibrators and proper vibration), curing and protection;

- Geometric tolerances - depending on the structural element; and
- Inspection classes - this standard defines inspection classes that, in accordance with the complexity of the structure, provide further instruction on which types of measures should be undertaken in order to fulfill quality requirements of the structure.

The main characteristic of BAS EN 13670:2011 is determination of execution classes according to works intended to be done. In fact, these classes define the level of quality control of works and types and frequency of in situ testing. BAS EN 13670:2011 defines three classes for execution:

- Execution class 1 should only be used for structures where consequences in case of failure are small or negligible and might be carried out by the operator that performed the work: this implies an inspection to be carried out on all work done – self inspection; [56]
- For structures in the Execution class 2, the inspection should concern at least concrete and reinforcement works for important structural members like columns and beams. There should be, in addition to the self-inspection, an internal systematic and regular inspection with fixed routines within the company that is performing the work that is an internal systematic inspection; [56]
- For inspection in Execution class 3, there may be required in addition an extended inspection according to national regulations and/or project execution specifications: this extended inspection may be performed by another company that is an independent inspection. In Execution class 3, the internal systematic inspection should include any concrete works of significance for the load bearing capacity and durability of the structure. [56]

Although the construction works for the WWTP were not assigned to any of these three classes, when analyzing terms given by the BAS EN 13670:2011, and comparing them to the tools of implementation of the QA program, it could be observed that these terms were mirrored in the QA program of the construction of the WWTP.

CONCRETE MIX DESIGN

Concrete mix design is the design of quantity of components for concrete – quantity of cement, water and aggregate are designed to be in a specific ratio with a goal to achieve required class of concrete. The typical concrete mix design may include the following:

- Input data about component materials with short description and quantities;
- Test records of each individual component; and
- Initial test records and conformity evaluation.

When discussing the significance of concrete mix design for the purpose of construction of the WWTP, the first characteristic that emerges is the existence of two separate concrete mix designs. From the aspect of concrete technology, concrete mix design for piles needed to be different in comparison to the concrete mix design for all other reinforced concrete structures.

Since the actual location of the WWTP had poor load bearing soil, the foundation design for the facilities was resolved by designing appropriate reinforced concrete piles. The average depth of appropriate load bearing soil was approximately 17 – 20 m, so the length of piles had to be the same. The diameter of the piles was 610 mm, and they were placed on axial distance of 3.50 m. The method for pile installation was relatively rarely used in the region. The total of 681 piles were installed by the full displacement method – FDP method.

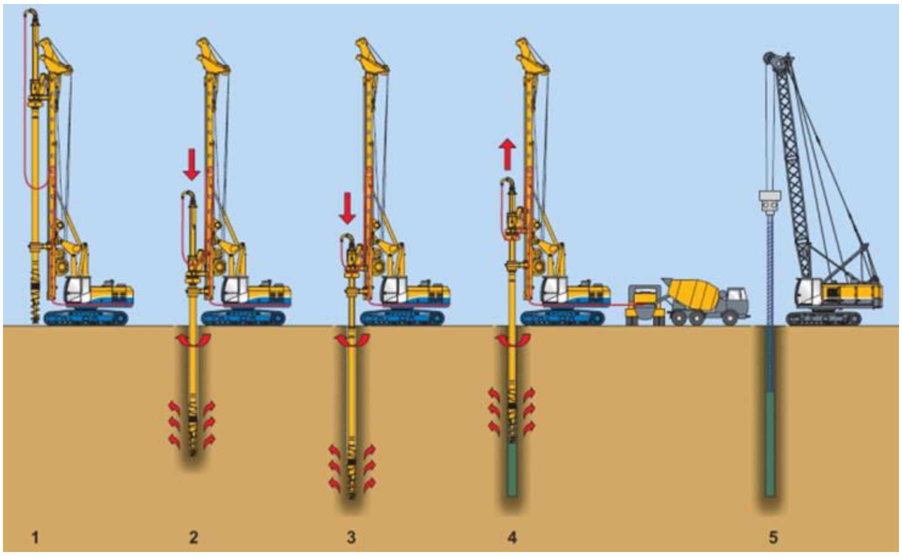


Figure 8 – Operating Sequence during Execution of FDP Piles [9]

Full Displacement Piles are bored cast in-situ concrete piles constructed by advancing a displacement boring tool into the ground with a rotary drilling rig using both torque and crowd force. The scheme of operating sequence is described in *Figure 8*.

Since the reinforcement cage for the pile was inserted into the fresh concrete using the technology of FDP Method, two concrete mix designs were necessary: one mix design for piles and the other one for structures of facilities. The main difference between these two concrete mixes is workability of concrete – concrete consistency.

Table 1 - Exposure Classes for Both Concrete Mixes [39]

Class designation	Description of environment	Informative examples where exposure classes may occur
No risk of corrosion attack		
X0	For concrete without reinforcement or embedded metal. All exposures except where there is freeze/thaw, abrasion, or chemical attack. For concrete with reinforcement or embedded metal. Very dry.	Concrete in buildings with very low air humidity.
Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity. Concrete permanently submerged in water.
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact; Many foundations
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2.
Chemical attack		
XA1	Slightly aggressive chemical environment	Concrete exposed to natural soil and ground water according to Table 2. of BAS EN 206:2014
XA2	Moderately aggressive chemical environment	Concrete exposed to natural soil and ground water according to Table 2. of BAS EN 206:2014
Freeze/thaw attack with or without de-icing agents		
XF3	High water saturation, without de-icing agent	Horizontal concrete surfaces exposed to rain and freezing

When analyzing the structure of the facilities of the WWTP, 22 out of 25 structures are completely reinforced concrete structures, while the remaining three facilities are RC frame structures with masonry infill.

The component materials used in the concrete mix for piles and other structures were designed in a relation to concrete class requirements and defined exposure classes. Concrete mix designs had to meet requirements of concrete classes C8/10, C16/20, C20/25, C25/30 and C30/37, with exposure classes X0, XC1, XC2, XC4, XA1, XA2 and XF3. BAS EN 206:2014 describes these exposure classes as shown in *Table 1*.

Since the test result records made for all of the mentioned concrete classes were quite extensive, this study would present data for concrete class C30/37, and in both concrete mix designs: for piles, and for concrete mix design for all the other structural elements.

Aggregate makes approximately three quarters of concrete mixture and it is considered as inert material in the mixture; however, it can sometimes affect the physical, thermal, and chemical properties of concrete.

The function of aggregates in concrete can be observed through fresh and hardened concrete. Namely, the grain size composition in fresh concrete mixture affects the consistency and cohesiveness of concrete. Discussing the hardened concrete, aggregate influences the strength of concrete, creep and shrinkage of the cement paste, and has an impact on rigidity, volume weight, and wearing resistance.

Figure 9 – Quarry of Aggregate for Concrete Mix for Piles [43]

Aggregate used in concrete mix for piles was crushed limestone from the stone quarry, with specific weight of 2780 kg/m^3, with the following grain size distribution:

- 0 – 4 mm - 55%
- 4 – 8 mm - not used
- 8 – 16 mm - 45%
- 16 – 32 mm - not used.

This bulleted list showed that only two out of four grain sizes were used in this concrete mix. The reasons for this specific concrete mix were supported by the following two facts:

1. Specificity of works on installation of piles (as it can be seen from *Figure 8*), the maximum aggregate grain size was limited to 16 mm in order to ensure unobstructed passage of concrete through concrete pump pipes and the hollow drill stem on the drilling machine; and
2. Highly viscous concrete mix was created by using only two types of fractions in order to avoid formation of voids in concrete during its placement, which would harm the integrity of the pile.

The grains size distribution of aggregate for mix design for piles is presented in the figure below.

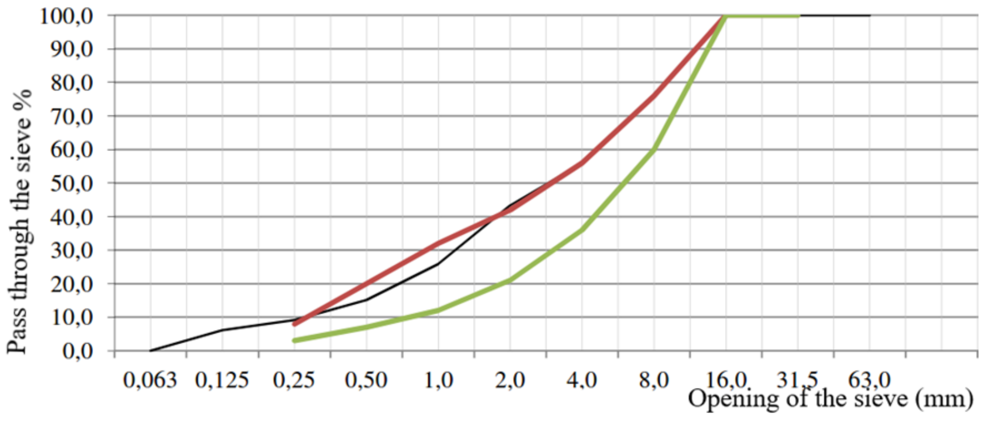


Figure 10 – Grain Size Distribution for Two Grain Size Aggregates used in the Concrete Mix for Piles [7]

Aggregate used in the concrete mix for structural elements of facilities of the WWTP was also crushed limestone from the stone quarry, with specific weight of 2830 kg/m^3, with the following grain size distribution:

- 0 – 4 mm - 50%
- 4 – 8 mm - 10%
- 8 – 16 mm - 40%
- 16 – 32 mm - not used.

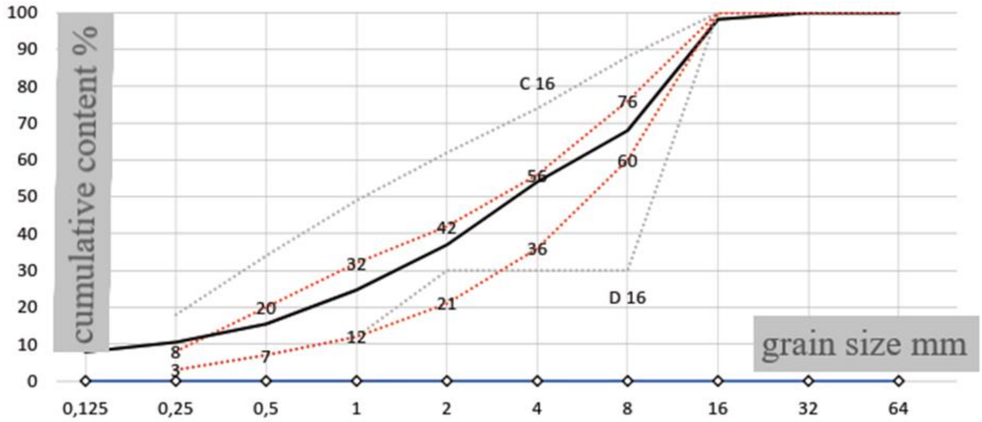


Figure 11 - Grain Size Distribution for Aggregates in Concrete Mix Design for other Structural Elements of Facilities of WWTP [13]

An additional reason for exclusion of the grain size 16-32 mm, was to ensure an easy pumping of concrete (especially for vertical tall elements) and to avoid any troubles that may be caused by larger aggregate particles in the concrete mix.

Figure 12 - Scheme of Concrete Pumping [49]

Water is one of the main ingredients in concrete. Water in the concrete mixture takes about 15 to 25% of the volume. Water in fresh concrete reacts with cement powder, producing hydration products and contributing to the incorporation of fresh mixture.

In order to improve the workability of concrete, the amount of water added to the concrete mixture is always greater than required for the hydration of cement; it has to be free of any harmful particles which could in any way affect or damage the properties of concrete, especially the quality of the reinforcement; for example, water impurities in the concrete mixture may have an effect on the consistency of concrete by slowing the binding process thus having a negative effect on the cement hydration. Water impurities can also cause excessive air intake, thereby reducing the strength of concrete. In general, potable water can also be used for mixing, but it can also contain specific minerals which don't have characteristics of drinking water. It can be said that water which, when shaken, does not create a foam, does not have a specific odor and color, is considered to be suitable for the concrete mix.

BAS EN 1008:2002 lists the following types of water:

- Drinking water: suitable for concrete, not necessary to be examined;
- Water used in the concrete industry obtained by renewal (i.e. water from rinsing): generally suitable for concrete, but the conditions set out in Annex A of the standard (for example, the additional dry weight in the purified water used in the concrete industry must be in concrete less than 1% of the total weight of the aggregate in the mixture);
- Groundwater: it can be suitable for concrete, but a previous check is needed;
- Natural surface water and water from industrial plants: it can be suitable for concrete, but a previous check is needed;
- Sea water or salt water: it can be used for unreinforced concrete, but it is not suitable for reinforced or prestressed concrete. It is necessary to adhere to the maximum allowed quantities of chloride for reinforced concrete and concrete with built-in steel reinforcements or metal parts;
- Wastewater: not suitable for concrete. [20]

For the purposes of concrete mixture preparation, water from public supply system was used in this project, since it met the requirements as a suitable concrete ingredient according to BAS EN 1008:2002.

The amount of water used in concrete mixture according to the concrete mix design was 183 kg in the concrete mix for piles and 175 kg in the concrete mix for the facilities of the WWTP.

The amount of cement used in both concrete mixes was 400 kg/m^3. Cement types used in the concrete mix design for the purpose of concrete mixtures for several concrete classes (according to different exposure classes) were as shown in the following table.

Table 2 - Cement Types used in Mix Designs

Structural element Cement type	Piles	Slabs and walls
CEM III/B 32,5N SR-LH	C30/37 for exposure classes XC2 and XA2	C30/37 for XC4, XA2, XF3
CEM II/B M(S-LL) 42,5N	-	C8/10 for X0 C16/20 for X0 C20/25 for X1 C25/30 for X2 C30/37 for XC4,XA1,XF3

Standard serial BAS EN 197 classifies cement types into five main groups:

1. CEM I – ordinary Portland cement;
2. CEM II – composite cement (moderate sulphate solubility and heat of hydration);
3. CEM III – metallurgical cement (early development of high strength);
4. CEM IV – puzzolanic (sulphate-resistant Portland cement of low heat hydration); and
5. CEM V – composite cement (high sulphate resistance).

In addition to the characteristics of the main types of cements, other constituents which slightly or significantly modify the main characteristics of cement, can be present depending on the purpose of application of cement. For example:

- Blastfurnace slag denoted by S;
- Silica fume denoted by D;
- Natural pozzolana denoted by P;
- Natural calcined pozzolana denoted by Q;
- Siliceous fly ash denoted by V;

- Burnt shale denoted by T;
- Limestone denoted by L;
- Composite cement denoted by M.

Considering compression strength, cements are classified in three groups based on the compression strength of the standard mortar after 28 days, as follows:

- 32.5 MPa;
- 42.5 MPa;
- 52.5 MPa.

For each class of standard strength, three classes of initial strength are defined:

- First class with ordinary initial strength – denoted by N;
- Second class with high initial strength – denoted by R;
- Third class with low initial strength – denoted by L.

Additionally, cement designation also contains information about the proportion of cement clinker, denoted by letters A (higher), B (medium) and C (lower). It should be noted that A, B or C values are not the same for each concrete type, i.e. the clinker content present in B class of cement type CEM II is 65 – 79%, while the class B of cement type CEM III has cement clinker content of 20 – 34%.

Ordinary Portland cements with sulfate-resistant properties are marked with letters SR. Ordinary Portland cements with properties of low-heat hydration are indicated by letters LH.

For example, designation of cement by CEM III/B 32,5N SR-LH has the following meaning:

- CEM III – indicates the main type of cement;
- B – medium cement clinker content (for the 20 – 34% type of cement);
- 32,5 – 32,5 MPa - the strength of the standard mortar after 28 days;
- N – ordinary initial strength;
- SR – sulfate-resistant; and
- LH – low-heat hydration.

Specifically, CEM III/B 32,5N SR-LH is sulphate resistant cement of low heat of hydration with 66-80% slag and 20-34% clinker (including binding regulator (natural gypsum). It is used for application in demanding construction projects where high sulfate resistance is required: concrete works in warm climate, structures in humid and aggressive environment rich in sulfates, bridge base, maritime and littoral objects, road works, irrigation systems, sewage and drainage systems. [50]

CEM II/B M(S-LL) 42,5N has high final strength and retains consistency for longer time, making the concrete mixture more workable and having excellent rheological properties of mortar. This is mixed Portland cement with 65-79% clinker and 21-35% of other constituents. This type of cement is the most widely used type of cement in various segments of construction execution.

Figure 13 - Sample of Hardened Concrete taken from the Construction Site of the WWTP

In comparison to ordinary cement type CEM II, the cement type III has lower heat of hydration, which is the significant advantage when performing concrete works in the summer. During the hot summer days, when performing concrete works, higher temperatures cause rapid evaporation of water from the concrete mixture. In the worst case scenario, excessive loss of water may cause cracks inside the concrete paste, a lot before concrete gains its maturity, which results in reduction of the strength of concrete. Low heat hydration of CEM III type contributes to lowering the risk for this scenario to occur by producing lower amount of heat during the process of hydration, thus not having significant impact in overall heat produced by the chemical processes of binding and hardening of concrete. During the cold weather, this type of cement is also preferred because it has the ability to gain high early strength, which makes for the second important advantage. A special attention has to be given to aggregates and water used in the mix, since, generally, concrete sets slowly in the cold weather, and frozen aggregates may prolong the setting time of concrete, which then opens the risk of freezing of water inside the pores and voids in concrete. This issue is easily solved by warming up aggregates and water during the cold weather.

There were two types of admixtures used in the concrete mixture:

- Plasticizer – 3 kg in the concrete mix for piles and 2.7 kg in the mix design for other structural elements;
- Air entraining admixture – 0.2 kg in the concrete mix for structural elements like slabs and walls.

Plasticizers are water reducing admixtures which can reduce the amount of water by 5-10%, or by 15–40% in which case they are superplasticizers. These admixtures reduce the amount of water, but do not affect the strength and durability of concrete. Basically, these admixtures are used on the construction site for the purpose of increasing the workability of concrete. When using these admixtures, water-cement ratio remains the same, while the viscosity of concrete increases, and therefore facilitates its placing and the concrete mixture is plasticized. Another purpose of this type of admixture can be to reduce the amount of cement in the concrete mixture without decreasing the strength of concrete, making concrete less expensive and more environmentally friendly. Dosing of such admixture is expressed as a percentage of the weight of the binding material in concrete, and there are two methods of dosing:

1. In dry condition, as powder (recommended); and
2. In the liquid state (it is necessary to know which quantity of powder is in the solution - from 30 to 40%).

It is preferred to add this admixture as a powder since there is a different amount of admixture powder in the liquid state.

The major drawbacks of superplasticizers are that they retard setting (especially in large amounts), cause more bleeding; and entrain too much air. The most common problem in the application of water reducers in concrete is incompatibility, which refers to the abnormal behavior of a concrete due to the superplasticizer used. Common problems include flash setting, delayed setting, rapid slump loss, and improper early-age strength development. These issues in turn affect the hardened properties of concrete, primarily strength and durability. Compatibility between cements and superplasticizers is affected by many factors, including cement composition, admixture type and dosage, and concrete mixture proportions. [17] According to the manufacturer's instructions, the amount of superplasticizer is recommended in the range of 0.2-1.2% (max 1.5%) from the total amount of binder.

Figure 14 - Concrete Before and After the Addition of Plasticizing Admixture [18]

Air-entraining admixtures absorb a specific amount of air into concrete. The drawn air is produced as the admixture causes foam during the mixing of concrete, and after that, the foam is trapped in concrete during the hardening of concrete. It is necessary to distinguish two types of pores in concrete:

- Targeted pores (from 50 to 200 μm) and
- Accidentally trapped air during mixing of concrete (3 mm).

There are many advantages in adding an air-entraining admixture to a concrete mixture:

- Improvements in workability of concrete, and consequently reduction in the amount of water in concrete;
- Improved ductility of concrete, due to the formation of concrete deformation space; and
- Improvements in impact resistance of concrete as air bubbles provide more space for deformation of concrete.

In addition to these advantages, the most important advantage of using this admixture in the concrete mix is its frost resistance. Actually, the small air bubbles in concrete provide space to release the pressure generated during the ice formation in the freezing process, which can prevent concrete from cracking and damage. [17]

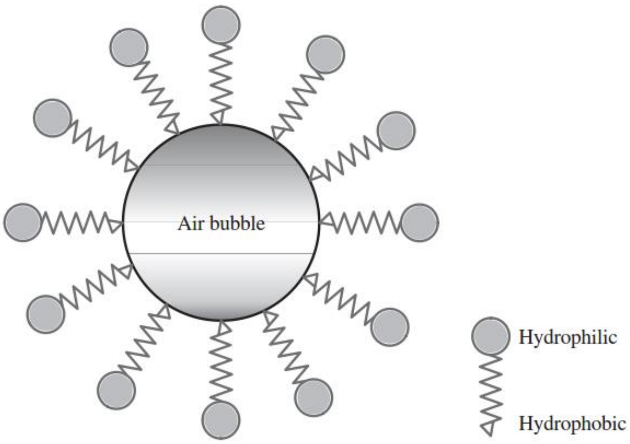


Figure 15 - Mechanism of Air Entraining [17]

When designing a concrete mix, it is necessary to prove that selected constituents and their ratio justify the production of concrete and that the designed concrete mix is achievable. This is done through the initial test, followed by the conformity test, in accordance to BAS EN 206:2014.

Definitions regarding the approval for production of concrete by BAS EN 206:2014 are as follows:

- Initial test: test or tests to check before the production starts how a new concrete family shall be composed in order to all specified requirements in the fresh and hardened state;
- Conformity test: test performed by the producer to assess conformity of the concrete;
- Evaluation of conformity: systematic examination of the extent to which a product fulfills specified requirements. [39] The initial test parameters that were tested regarding concrete mixture production, included:
  - ✓ Compressive strength which had to fulfill two criteria:

$$f_{\mathrm{cm}} \geq (f_{\mathrm{ck}} + 2\sigma_{\mathrm{oc}}) \tag{1}$$

$$f_{ci} \geq (f_{ck} - 4\ MPa) \tag{2}$$

where

f_{cm} – mean compressive strength of concrete;

f_{ck} – characteristic compressive strength of concrete;

f_{ci} – individual test result for compressive strength of concrete;

σ_{oc} – estimate for the standard deviation of population.

- ✓ Concrete consistency, which had to fulfill the following criteria:

$$S5 + 30 \geq S_{\max};\ S_{\min} \geq S5 - 20 \quad (3)$$

where

S5 – consistency class *S5* expressed by slump;

S_{max} – maximal slump;

S_{min} – minimal slump;

- ✓ *w/c* ratio, with the criteria:

$$w/c + 0{,}04 \geq w/c_{\min} \quad (4)$$

where

w/c – water/cement ratio.

The test report from the authorized laboratory showed that the initial concrete production criteria were fulfilled by the concrete manufacturer, and therefore concrete plant was capable of supplying construction site of the WWTP with concrete during the entire period of concrete works. In order to additionally support the capacity of concrete plant to continuously produce required amount of concrete, testing of production capabilities of the concrete plant were also performed; these testing obtained data about functionality of the concrete plant, such as:

- Characteristics of materials used;
- Functionality characteristics, technical data and condition of equipment;
- Data on precision of measurement and dosing devices;
- Time flow of work cycle;
- Homogenization capability of the mixer, which included evaluation of test results of the concrete mixture homogeneity and evaluation of test results of concrete mixture components dosing;
- Theoretical capacity of concrete plant; and

- Conclusion of authorized laboratory on production capacity of the required concrete.

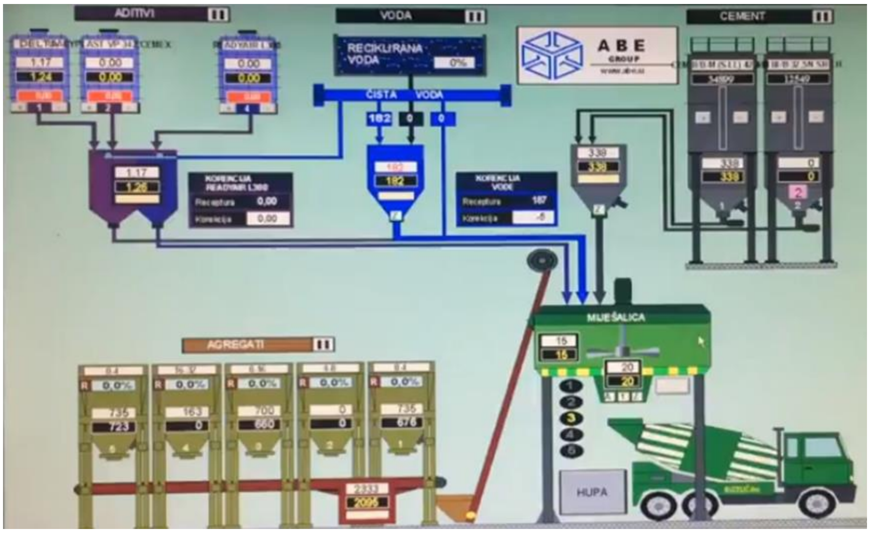


Figure 16 - The Integrated Technological System of Concrete Production in the Concrete Plant [43]

By examining the test results regarding the production capacities of the plant, it was concluded that the concrete manufacturing plant was capable to supply the construction site with concrete of required quality and required quantity.

TESTING OF FRESH CONCRETE

Before being placed in situ, concrete approved through the set of examinations in accordance to the conformity criteria of BAS EN 206:2014, needs to be subjected to further examination. These tests are part of identity testing of concrete. Identity testing represents the criteria which prove that concrete transported from the concrete plant to the construction site is of the same characteristics as concrete that satisfied conformity criteria in the concrete plant.

Tests for identity criteria could be grouped into two categories:

- Tests performed on the fresh concrete mixture, which are drawn from BAS EN 12350 series; and
- Tests performed on the hardened concrete, which are drawn from BAS EN 12390 series.

Tests performed on the fresh concrete mixture on the construction site of the WWTP included:

- Sampling of concrete for testing according to BAS EN 12350-1:2010;
- Temperature measurement of concrete according to JUS U.M1.032,
- Consistency testing according to BAS EN 12350-2:2010 and BAS EN 12350-5:2010;
- Testing of entrained air according to BAS EN 12350-7:2010; and
- Unit mass of concrete.

Figure 17 - Equipment for Testing of Fresh Concrete on the Construction Site of the WWTP

During the first step of the testing, the sampling can be done in two ways, as described in the BAS EN 12350-1:2010; the samples can be taken either as composite samples or as spot samples. Defined by EN 12350-1:2010, the composite sample refers to the quantity of concrete, consisting of number of increments distributed through a batch or mass of concrete thoroughly mixed together, where batch represents (in the case of transported concrete from the concrete plant to the construction site of the WWTP) quantity of fresh concrete which is conveyed and ready-mixed in a truck mixer when the load requires more than one cycle of a batch mixer, or continuously more than one minute in concrete mixer. Spot sample is the quantity of concrete taken from the part of a batch or mass concrete, consisting of one or more thoroughly mixed together increments.

The quantity of concrete required for sampling for the purposes of test performance has to be 1.5 times the estimated required quantity. When obtaining the composite sample from ready mixed concrete truck, a minimum of four increments are recommended.

The temperature measurement of fresh concrete mixture is performed either in the concrete mixer or in the concrete placement spot, as well as in the newly cast structural element. The concrete temperature value is the value calculated as the mean arithmetic value obtained from by thermometer on 5 measurement points of 5–10 cm below the concrete surface.

Figure 18 - Measurement of Concrete Temperature on the Construction Site of the WWTP

According to the QA program for construction of the WWTP, the temperature of concrete and the outside temperature had to be measured and recorded for every concrete delivery.

During concrete installation, the focus is on the workability of concrete. The workability of concrete is the measure of concrete consistency which depends on the geometry of the structural element to be cast. Consistency of concrete primarily depends on the water/cement ratio. In addition to the w/c ratio, concrete consistency may be improved in different ways. For example, in the case of transported concrete mixtures, consistency can be improved by turning the concrete mixer to higher rotational speed to prevent early concrete bonding and to exclude the possibility of segregation in concrete mixture by constantly maintaining the homogenous mixture. Another way of improving the concrete mixture consistency is to apply the admixture – plasticizer. Addition of this admixture should be done carefully, since overdosing could end in segregation of concrete. Also, when another admixture exists simultaneously, like the air entraining admixture, the influence of one admixture to another one becomes quite significant. Combining plasticizer and air entraining admixture simultaneously can result in improved workability and diminished risk of segregation and bleeding. On the other hand, improper ratio can result in higher content of micro pores, which results in reduction of concrete compressive strength.

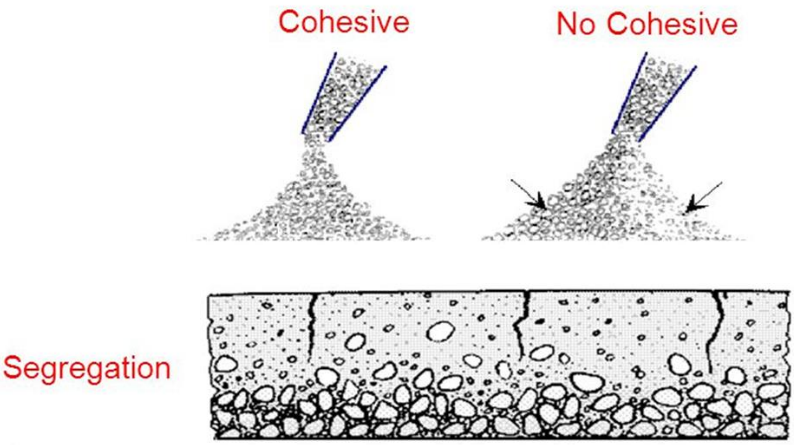


Figure 19 - Segregation in Concrete [51]

Narrow elements, especially those highly reinforced due to larger internal forces that appear in the element are difficult to concrete properly, since vibrator needle is obstructed and cannot properly penetrate into concrete layers. To avoid these situations, BAS EN 206:2014 defined the slump classes which were necessary to be determined in the design of concrete mixture.

For the purposes of concrete works execution on the WWTP, two categories of slump classes were present. These two categories were determined through the concrete mix designs: for reinforced concrete piles and for other structural elements (columns, walls and slabs).

The concrete mixture for piles needed to be of high consistency class. The reason for this was to provide the integrity of the piles, without any discontinuities in concrete, so that no voids or pockets in concrete would form, resulting in secure transfer of load to the load bearing soil. This type of consistency was tested by the flow table test according to BAS EN 12350-5:2010.

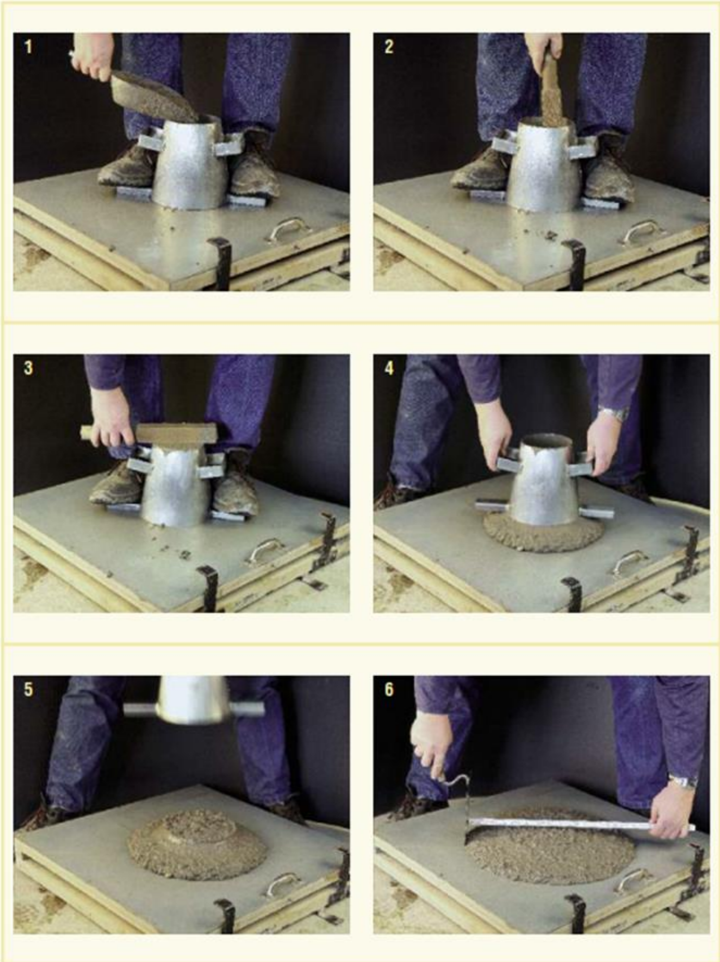


Figure 20- Procedure of Table Flow Test [18]

The procedure for consistency testing according to BAS EN 12350-5:2010 considers the following steps:

- The table flow test determines consistency by measuring the spread of concrete on a flat plate that is subjected to twitching;

- The width of the spreading plate is 700 ± 2 mm x 700 ± 2 mm, to which a taper cone is placed in the center, a base dimension of 200 ± 2 mm, top of the base 130 ± 2 mm and a height of 200 ± 2 mm;

- The mold is placed in the middle of the board, and the legs of the molding pad are pressed, and then the mold is filled in two layers of approximately the same height. Each layer is charged with 10 bumps with a bar. When the mold is filled with a concrete mix to the top, the top surface is aligned and then after one minute with the pedal mold slowly lifting, where the concrete more or less shrinks. Then, the upper panel with the handle 15 times slowly rises and descends to freely fall from a height of 4 cm. Then the diameter of the expanded concrete is measured parallel to the sides of the board d_1 and d_2. [3]

The consistency class is obtained by the following equation:

$$F = \frac{d_1 + d_2}{2} \tag{5}$$

where

F – Flow diameter tested in accordance with EN 12350-5:2010.

Based upon *F*, consistency class is obtained according to the following table:

Table 3 - Consistency Classes by Table Flow Test [39]

Class	Flow diameter in mm
F1	≤ 340
F2	350 – 410
F3	420 – 480
F4	490 – 550
F5	560 – 620
F6	≥ 620

The consistency class, specified by the table flow test, was defined for the concrete mix for piles, since its consistency was not measurable by standard slump test. Frequency of taking the samples and testing the consistency was every 20 m^3 of delivered concrete until the equalization of the obtained results. The actual tests showed that every concrete batch tested in situ prior to concrete placement had the designed value of consistency (*F5*).

Another type of consistency testing applied during the construction of the WWTP in Bihać was slump test. Slump test is the most performed test since it is adequate for ordinary types of concrete and execution of concrete works. The main characteristics and steps of performing the slump test are as follows:

- The test is sensitive to changes in consistency in concrete, which correspond to a slump value of between 10 mm and 21 mm. Outside these limits, the measurement of the settlement may be inappropriate and other methods of determining consistency should be taken into account (such as table flow test);

- The test is not suitable when the maximum size of the aggregate in concrete is greater than 40 mm;

- The mold for performing the test must be in a form of a hollow taper cone and have the following measures:
  - ✓ base diameter: (200 ± 2) mm;
  - ✓ diameter of the tip: (100 ± 2) mm;
  - ✓ height: (300 ± 2) mm;

- Fresh concrete is compacted into a mold shaped as a truncated cone. When the cone is lifted upwards, the distance from the settled concrete gives a measure of consistency of concrete;

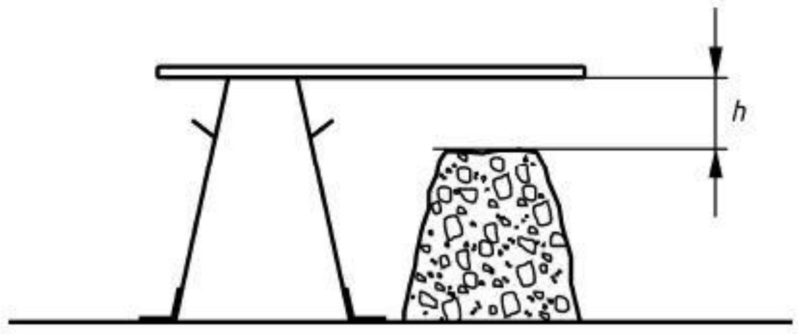


Figure 21- Measurement of the Slump [57]

- The mold should be filled with concrete in three layers, each approximately one third of the height of the mold. Compaction of each layer is done with 25 rod strokes all over its depth, so that the impact only penetrates the other layer below.

Figure 22 - Concrete Slump Test on the Construction Site of the WWTP

Concrete consistency values obtained by the slump test are grouped in the following classes according to BAS EN 206:2014, as shown in the table below.

Table 4 - Consistency Classes obtained by the Slump Test [39]

Consistency class	Slump values in mm
S1	10 - 40
S2	50 - 90
S3	100 - 150
S4	160 - 210
S5	≥ 220

Concrete subjected to slump test during the concrete placement was for structural elements, that is, columns, slabs and walls. Slump class S4 was required for columns and walls because of their specific dimensions – narrow and high cross sections, while concrete of consistency class S3 was required for slabs.

As defined by the concrete mix design and the QA program, the frequency of concrete sampling was performed on every 20 m^3 of concrete and in such situations where doubts about certain deviations existed.

The significance of entrained air in concrete was explained in earlier considerations of this study involving air entraining admixture. Air entrainment was tested according to BAS EN 12350-7:2010 standard.

Figure 23 - Testing of Air Content in Concrete on the Construction Site of the WWTP

The procedure of testing the air entrainment begins with sampling of concrete from the total volume of concrete that is between 10 and 90 % of the delivered batch. Generally, concrete sampling for testing should be done after at least a minimum amount has been unloaded from the concrete mixer (that is 10%) in order to have a representative sample.

When the sample is obtained, the measuring bowl is filled with three layers, each layer been consolidated 25 times by rodding. When rodding the first layer, the rod must not strike forcibly the bottom of the bowl. When rodding the other layers, the rod should just penetrate into previous layer. Besides rodding, the sides of the bowl must be tapped 10 or more times to remove cavities and large air bubbles in the concrete mixture (which could be done with a rubber hammer). After filling the last, third layer, the surface of the concrete mixture in the measuring bowl must be fully leveled with the edge of the bowl.

After performing these steps, the bowl cap is put on the bowl; when cover is clamped in place, a pressure-tight seal is achieved. After closing the bowl, water is then filled through one petcock until the water coming out from another petcock no longer has bubbles, which means that the trapped air has been expelled. The air is then pumped beyond the initial calibrated pressure point, so it can be stabilized back to the initial pressure, after which the closing of petcocks follow. Then, by opening the appropriate valve, the air is released into the chamber. The valve stays open until the gauge needle stabilizes while gently tapping the gauge. By releasing the valve, the air content can be recorded.

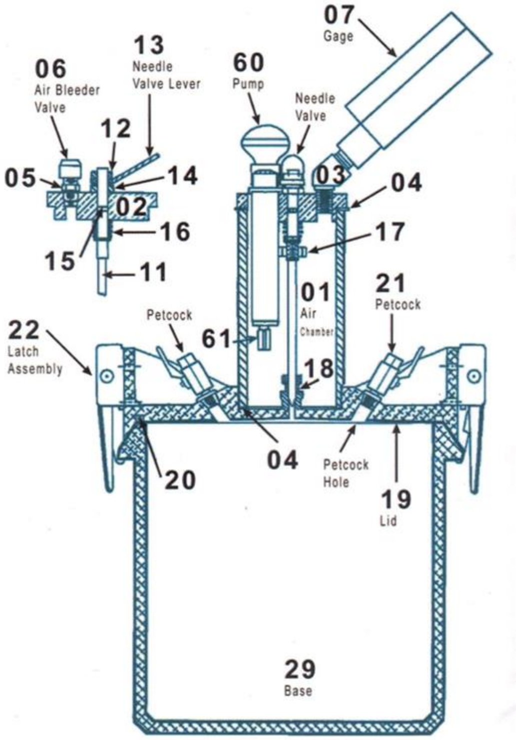


Figure 24 - Cross Section of Concrete Air Meter [54]

Unit mass of concrete is measured in situ by using proper laboratory scale and measurement bowl that is used in examination of air content in the concrete mixture. The unit mass is measured in the way that the mass of an empty bowl is measured first. When its mass is recorded, the bowl is then filled with concrete mixture by following the procedure for air entrainment. After filling the bowl, it is brought to laboratory scale to weigh the bowl filled with concrete. The unit mass is obtained through the following equation:

$$\rho = \frac{m_{\text{concrete+bowl}} - m_{\text{bowl}}}{V_{\text{bowl}}} \tag{6}$$

where

$m_{\text{concrete+ bowl}}$ - mass of bowl with concrete;

m_{bowl} - mass of bowl;

V_{bowl} - volume of bowl.

Figure 25 - Measurement of the Unit Mass of Concrete [55]

TESTING OF HARDENED CONCRETE

Tests performed on the hardened concrete on the construction site of the WWTP were:

- Testing of the compressive strength of concrete according to 12390-3:2010;
- Testing of depth of water penetration into concrete under pressure according to EN 12390-8:2010;
- Testing of concrete frost resistance according to JUS U.M1.016.

Compressive strength of concrete is the main aspect when designing concrete structures. The requirements that regulate the quality control of concrete are given in BAS EN 206 in the framework of identity testing. Identity testing is the examination to evaluate whether concrete delivered to construction site corresponds to the concrete produced in the concrete plant, which is evaluated by the conformity criteria. The identity is determined as the average of *n* samples taken from the defined volume of concrete.

The identity criteria for the purpose of identity testing of concrete compressive strength on the construction site of the WWTP is shown in the following table.

Table 5 - Identity Criteria of Concrete Compressive Strength [39]

The number of results n from the defined volume of concrete	**Criteria 1**	**Criteria 2**
	Mean of n results (f_{cm}) N/mm^2	Any individual test result (f_{ci}) N/mm^2
1	Not applicable	$\geq f_{ck} - 4$
2 to 4	$\geq f_{ck} + 1$	$\geq f_{ck} - 4$
5 to 6	$\geq f_{ck} + 2$	$\geq f_{ck} - 4$

The procedure for testing the compressive strength of concrete is as follows:

- Samples are tested to compression failure according to the BAS EN 12390-4:2003. The maximum load applied to the sample is recorded and the compressive strength of the concrete is calculated;
- The testing sample is a 15/15/15 cm cube in accordance with BAS EN 12390-1:2013;
- A constant load speed is selected in the range of 0.6 ± 0.2 MPa/s. After applying the initial load, which does not exceed about 30% of the failure load,

the load is applied to the sample without impact and continuously increased with the chosen constant speed of ± 10% to failure; The compression strength of the concrete is calculated according to the expression: [4]

$$f_c = \frac{F}{A_c} \tag{6}$$

where

f_c – compressive strength of the individual cube in MPa;

F – maximum failure force in concrete in N;

A_c – cross sectional area of concrete in mm^2.

Figure 26 – Sampling and Compressive Strength Testing (WWTP)

Defined by the concrete mix design and the QA program, the concrete compressive strength testing was conducted as follows:

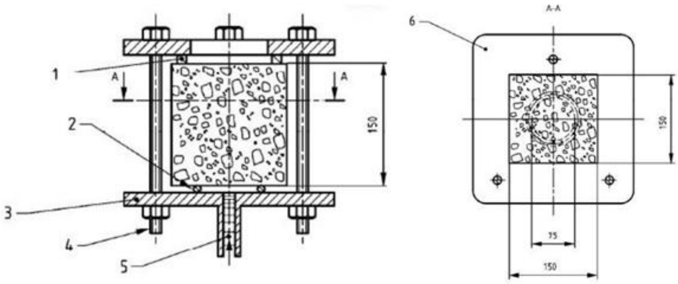
- Minimum one sample every day of concrete works for each type of concrete;
- Minimum one sample for each 100 m^3 of installed concrete;
- Minimum one sample a day for structural elements significant for the safety of the structure, regardless of the concrete amount built in.

Figure 27 - Curing of Concrete Cubes on the Construction Site of the WWTP

Testing the depth of water penetration into concrete is significant for water tight facilities that are intended to hold water. In fact, water tightness of concrete represents a value that shows the limited depth of penetration of water into concrete. The facilities of the WWTP, by the design requirements, are intended to hold water with the maximum allowed water penetration of 3 cm. Thus, the concrete cover for structural elements for such facilities of the WWTP is 5 cm.

The procedure of testing the water tightness of concrete is as follows:

- The principle is that water is applied under pressure to the surface of hardened concrete. The specimen is then split and the depth of penetration of the waterfront is measured;
- The test specimen, of given dimensions, shall be placed in any suitable equipment in such a manner that the water pressure can act on the test area and the pressure applied can be continuously indicated;
- The specimen in this particular case is cubic, with dimension of the surface 150/150 mm;
- The test shall be started when the specimen is at least 28 days old. The water pressure should not be applied to a trowelled surface of a specimen. The specimen is placed in the apparatus and applied a water pressure of 500 ± 50 kPa for 72 ± 2 h. During the test, it is required to periodically observe the appearance of the surfaces of the test specimen not exposed to the water pressure to note any water presence. If leakage is observed, then consideration of the validity of the result and record is necessary;



1. – packing piece
2. – sealing ring
3. – screwed on plate
4. – screw-threaded rod
5. – water under pressure
6. – screwed on plate

Figure 28 - Test Arrangement for Depth of Water Penetration into the Concrete [39]

- After the pressure has been applied, the specimen should be split in half, perpendicularly to the face on which the water pressure was applied. When splitting the specimen, and during the examination, the face of specimen exposed to the water pressure should be placed on the bottom. As soon as the split face has dried to such an extent that the water penetration front can be clearly seen, the water front on the specimen can be marked;
- Measuring the maximum depth of penetration, expressed in mm, is the test result. [39]

Figure 29 - Device for Water Penetration Test

In addition to testing of concrete water tightness in the authorized laboratory, some additional tests had to be conducted to verify the overall water tightness for each facility intended to hold water. This testing consisted of two phases:

- Saturation phase: phase of saturation of walls and slab with water for 7 days; and
- Testing phase: phase of testing by measurement of water level in the tanks on daily basis for 5 days, where water loss (level) should meet limit criteria of permissible water loss after these 5 days. [15]

Figure 30 - Activated Sludge Tank and Final Sedimentation Tanks Filled with Water for Water Tightness Testing

This test showed that some water loss occurred through walls, most probably as a consequence of inadequate vibration of walls during casting of concrete in spite of full concrete quality control. However, water loss was below than maximally permissible value for such test. [15]

The importance of entrained air and thusly formed pores is explained earlier. To improve the durability of concrete to frost, these pores are intentionally created. As it is known, the volume of ice is approximately 10% bigger than the volume of water. When water enters the pores in concrete and freezes on lower temperatures, it has the space for volume enlargement without breaking the surrounding concrete matrix and therefore without the formation of cracks in concrete that may result in fracture.

Additionally, to provide designed concrete compressive strength and necessary frost resistance, the Rulebook on technical regulations for construction products to be built in concrete structures [42] prescribe the allowed range of amount of pores, which is dependent on the maximum grain size in the concrete mixture. Since the maximum grain size in the mixture is 16 mm, the amount of pores by air entraining is from 3% to 5%.

Since BAS CEN/TS 12390-9:2007 - Testing hardened concrete, Freeze-thaw resistance, has been withdrawn, there was no valid European standard, thus the testing was done according to valid national standard JUS U.M1.016.

Concrete labels for freeze-thaw resistance are: M-50, M-100, M-150, M-200, M-250, where numbers present the largest number of freezing and thawing cycle that specimens must resist according to the procedures established by this standard, at which point the amount of average strength at pressure from freezing specimens must be 75% of the average strength of non-freezing specimens of equivalent age (etalons) in a destructive method, and the procedure is following:

- The specimens must be of dimensions 150 mm or 200 mm, climate controlled room or chamber with a temperature of (20 ± 2) °C and compressive strength measurıng device;
- One cycle consists of freezing of steady temperature of -20 °C ± 2 °C in the duration of 4 hours and thawing in water at temperature of +20 °C ± 3 °C also for 4 hours. Breaks during thawing is not allowed;
- On the day of testing of freeze-thaw resistance, compressive strength must be tested at pressure of 3 etalon E_0;
- Equivalent age T_e due to slow growth of pressure for freezing time is determıned wıth equation:
    - ✓ For 3 cycle freeze-thawing in 24 hours:

$$T_e = a + 0{,}2n\ (cycle\ 4h + 4h) \tag{7}$$

- ✓ For 2 cycle freeze-thawing in 24 hours:

$$T_e = a + 0{,}35n\ (cycle\ 4h + at\ leatst\ 4h\ for\ the\ first\ thawing) \tag{8}$$

- ✓ For 1 cycle freeze-thawing in 24 hours:

$$T_e = a + 0{,}8n\ (cycle\ 4h + 20h) \tag{9}$$

where:

a – specimens age (in days) on the beginning of freezing;

n – cycle numbers alternate freeze-thaw before compressive strength testing. [40]

By non-destructive method, the concrete dynamic modulus of elasticity of freezing specimens has to be at least 75% of concrete dynamic modulus of elasticity which have non-freezing specimens saturated with water.

- The first dynamic modulus of elasticity measuring is done on the day of specimens' preparation. The next measurements of concrete dynamic modulus of elasticity are done after every 25 cycle of freeze-thaw;
- The degree of concrete resistance to frost is determined by comparing the concrete dynamic modulus of elasticity of last measuring to concrete dynamic modulus of elasticity of second measurement. [40]

Conformity criteria by JUS U.M1.016 was satisfied when there was less than 25% of concrete strength change, tested with the destructive method that was conducted on samples from the WWTP construction site.

Figure 31 - Device for Testing of Freeze-Thaw Resistance of Concrete

TESTING OF STEEL REINFORCEMENT

When considering the quality control of the reinforced concrete, the quality control of the reinforcement is inevitable and integral part of QA process. The standard that gives guides and procedures on testing the reinforcement is BAS EN 10002-1:2002 – Metallic materials – Tensile testing – Part 1: Method of test at ambient temperature.

In order to meet main requirements, such as weldability and duration, reinforcement steel needs to be of certain chemical composition in corresponding ratio. This chemical composition includes elements shown in the Table 1, the maximum amount of elements allowed to be in the composition. [5]

Table 6- Chemical Composition of Reinforcement Steel [1]

Chemical composition (% per mass)							
Technical class	C	Mg	S	P	N	Cu	Equivalent amount of C
B500B	**0,16-0,22**	**0,40-0,60**	**0,050**	**0,050**	**0,012**	**0,80**	**0,50**

The classification of the reinforcement steel is done by the ductility criteria in classes A, B or C, according to BAS EN 10080:2007. The characteristics of each class is shown in the following table.

Table 7 - Reinforcement Steel Characteristics according to BAS EN 10080:2007 [19]

Class and label of steel	**B500A (1.0438)**		**B500B (1.0439)**		**B450C (1.04..)**
Shape of product	Coiled	Rods/bars	Coiled	Rods/bars	Coiled
Nominal diameter d [mm]	4-16	6-40	6-16	6-40	6-16
Yield strength Re [MPa]	≥500		≥500		≥450
Ratio of tensile strength and yield strength R_m/R_e	≥1,05[1)]		≥1,08		≥1,15 ≤1,35
Total elongation A_{gt} [%]	≥2,50[2)]		≥5,00		≥7,50
[1)] $R_m/R_e \geq 1{,}03$ for d=4,0 to 5,0 mm **[2)] $A_{gt} \geq 2\%$ for d=4,0 to 5,0 mm**					

The classification of the reinforcing steel can be distinguished by the appearance of the reinforcement. Namely, the rib arrangement on the reinforcement rod differs from type to type, which can be seen from the figure below.

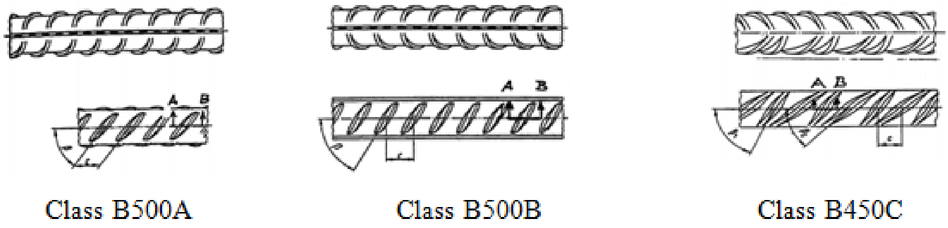


Figure 32 - Reinforcement Appearance according to Ductility Classes [52]

When analyzing the label of the reinforcement, according to EN 10027-1:2007 - Designation systems for steels – Part 1: Steel names, first capital letter B represents the label of steels for reinforcing concrete, followed by number 500 or 450, which represents characteristic yield strength in MPa for the smallest dimensional range.

Figure 33 - Installation of the Reinforcement on the Construction Site of the WWTP

The tensile test of reinforcement bars involves straining a test sample in tension, generally to fracture, for the purpose of determination of one or more mechanical properties. [22]

In the reinforcement tensile test, the sample is treated uniaxial until fracture occurs. Before the loading of the reinforcement bar, the cross sectional area S_o and original gauge length L_o are recorded. After placing the sample in the jaws of the machine for tensile test performance, the tensile force then can start acting on the sample, where, after certain amount of load can be observed the reduction in rod diameter, as well as its elongation.

After sample fracture, final gauge length L_u and minimum diameter S_u are measured. For the purpose of determination of characteristics of reinforcement, following values are considered: [46]

- Gauge length (L) - length of cylindrical or prismatic portion of the test piece on which elongation is measured at any moment during the test [m];
- Original gauge length (L_o) - gauge length before application of force [m];
- Final gauge length (L_u) - gauge length after rupture of the test piece [m];
- Elongation - increase in the original gauge length at the end of the test;
- Ductility – percentage elongation after fracture (A_{gt}) - permanent elongation of the gauge length after fracture, expressed as the percentage of the original length:

$$A_{\mathrm{gt}} = \frac{L_u - L_{\mathrm{o}}}{L_{\mathrm{o}}} \quad [\%] \tag{10}$$

- Extension – increase of the original length at a given moment of the test;
- Percentage reduction of area (Z) - maximum change of cross sectional area, which was occurred during the test, expressed as a percentage of the original cross-sectional area;
- Original cross-sectional area (S_o) of the test piece;

$$Z = \frac{S_{\mathrm{o}} - S_{\mathrm{u}}}{S_{\mathrm{o}}} \quad [\%] \tag{11}$$

- Maximum force (F_m) - the greatest force which the test piece withstand during the test [N];
- Stress (σ) - force at any moment during the test divided by the original cross-sectional area (S_o) of the test piece;

$$\sigma = \frac{F}{S_{\mathrm{o}}} \quad [MPa] \tag{12}$$

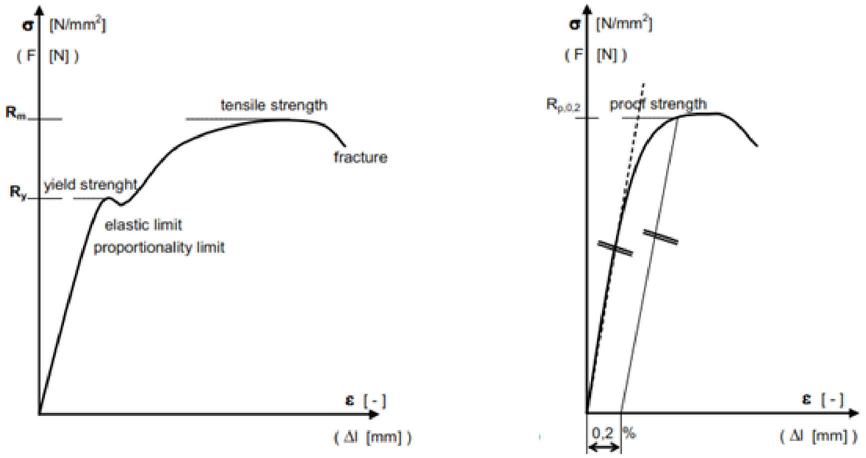
- Tensile strength (R_m) - stress, corresponding to the maximum force F_m:

$$R_{\mathrm{m}} = \frac{F_{\mathrm{m}}}{S_{\mathrm{o}}} \quad [MPa] \tag{13}$$

- Yield strength (R_y) – when metallic material exhibits a yield phenomenon, a point is reached during the test at which plastic deformation occurs without any increase in the force:

$$R_y = \frac{F_y}{S_o} \quad [MPa] \tag{14}$$

- Proof strength (R_p) – stress at which extension is equal to a specified percentage of the gauge length. The symbol used is followed by a suffix giving the prescribed percentage, for example $R_{p,0,2}$.



a) Steel with yield point b) Steel with proof strength $R_{p,0,2}$

Figure 34 - Stress-Strain Diagram of Steel [46]

Since the reinforcement used on the construction site of WWTP was B500B, with the diameter Ø8, Ø10, Ø12, Ø14, Ø16, Ø19, Ø20, Ø22, Ø25 and meshes Q335, the minimum yield strength is 500 MPa.

Required tensile strength, corresponding to obtained yield strength, is given by the ratio of tensile strength and yield strength R_m/R_e to be minimum 1.08. Total elongation should be equal to or greater than 5%. [5]

TEST RESULTS AND ANALYSIS

Following graphs and diagrams contain and present results of the conducted tests according to procedures from BAS EN 12350 and BAS EN 12390 series during the construction of the WWTP. These test results are presented for three structural elements: piles, foundation slab and walls. Test results for quality control of reinforcement are also shown in this chapter.

The graphically presented results in *Figures 35-37* are results of identity testing of the concrete compressive strength for concrete class C 30/37 during the construction of the WWTP. Blue lines on the diagrams present the mean values of test results of the concrete compressive strength throughout the period of conduction of tests, while the green line is the minimum value. For easier interpretation of diagrams, the compressive strength of 37 MPa (C 30/37) is presented with a black line, identity criteria 1 with pink line, and identity criteria 2 with red line.

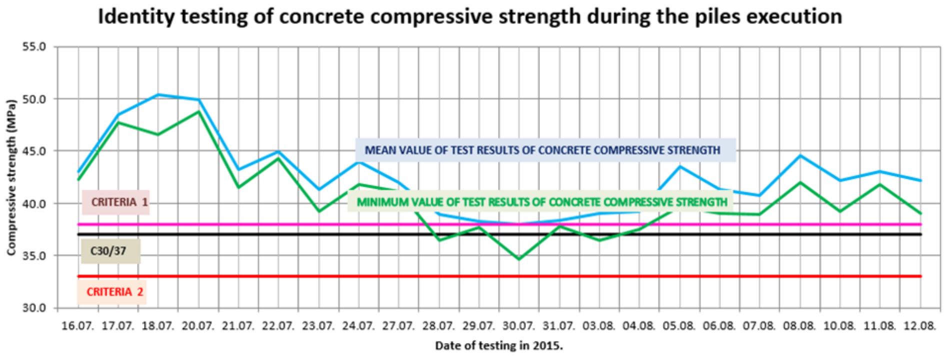


Figure 35 - Test Results of Identity Testing of Concrete Compressive Strength of RC Piles [4]

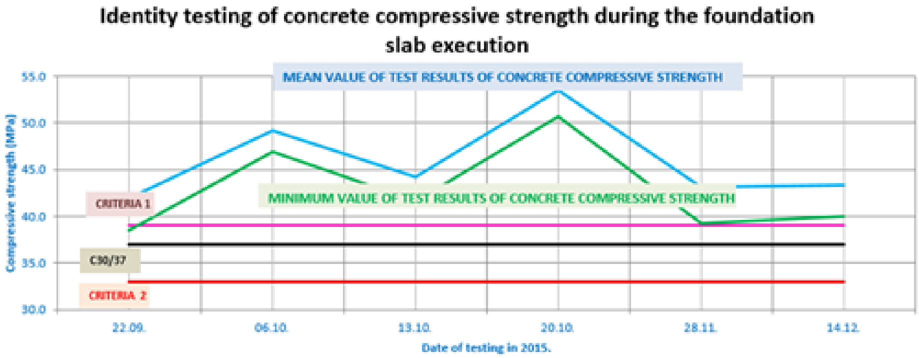


Figure 36 - Test Results of Identity Testing of Concrete Compressive Strength of RC Mat Foundation [4]

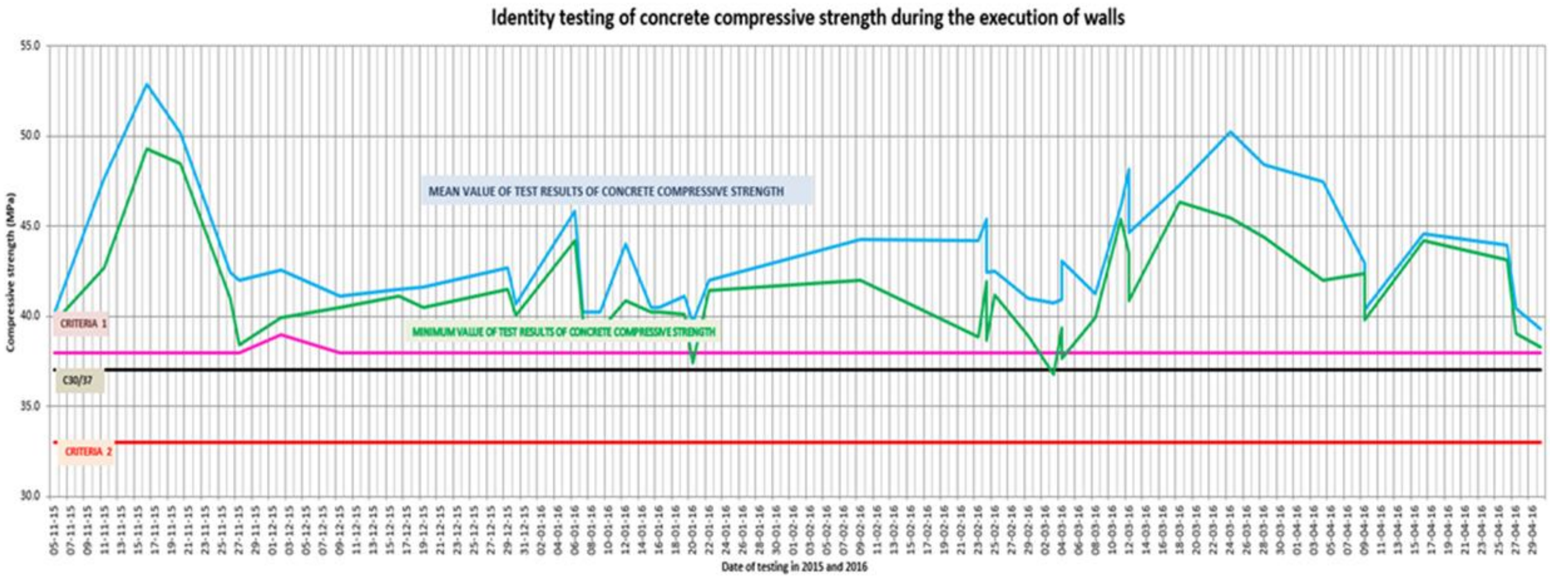


Figure 37 - Test Results of Identity Testing of Concrete Compressive Strength for RC Walls [4]

According to these graphs, we can observe the following:

- The mean values of the compression strength of concrete for all three structural elements are greater than the value of the identity criterion 1; [4]
- The minimum values of the compression strength of concrete are greater than the value of the identity criterion 2; [4]
- Based on these findings, it is estimated that the concrete compressive strength of all three structural elements is identical to the declared characteristic strength according to BAS EN 206: 2014;
- The tested concrete belongs to a complying set, and satisfies the requirements regarding of the concrete class C 30/37; [4]

Consistency testing of fresh concrete mixes was an integral and continuous activity of the Q/A program during concrete works at the construction of facilities of the WWTP in Bihać. *Figure 38* shows that minimum and maximum values of concrete consistency for piles, tested with table flow test method, were within the limits of the permissible consistency class F5 during the whole period of piles installation.

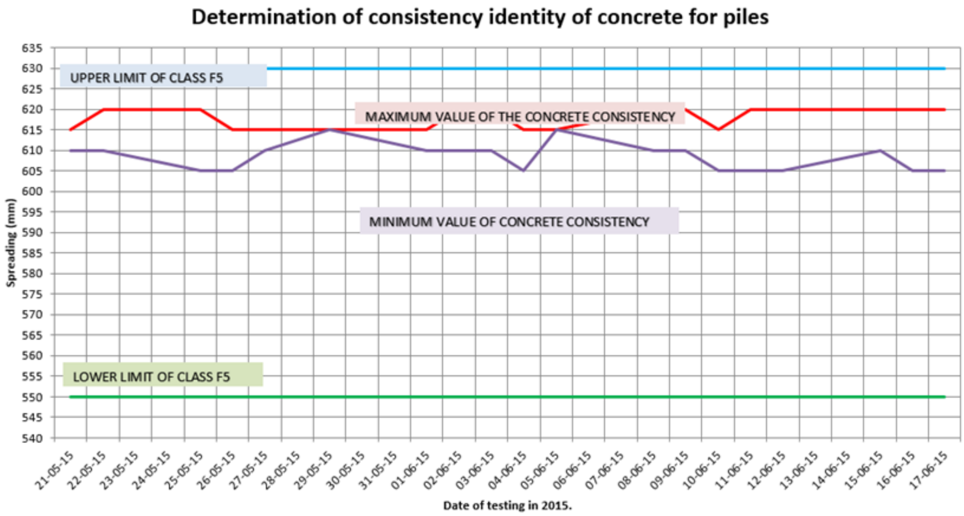


Figure 38 - Test Results of the Concrete Consistency Testing for Piles [3]

Figure 39 shows that the minimum value of consistency of concrete for the mat foundation, tested by the slump method, on September 15, 2015, was 130 mm, which was below the permissible lower limit of the consistency class S4, that is 150 mm. In addition, the maximum value of consistency of concrete for the mat foundation, on the same date, but tested on different delivery was 250 mm, which was above the permissible upper limit of consistency class S4, which is 220 mm. [3]

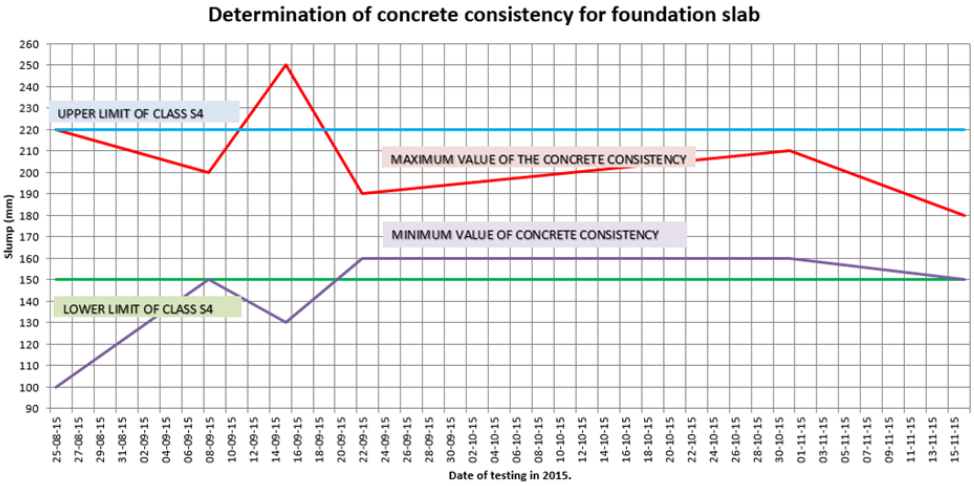


Figure 39 - Test Results of Concrete Consistency Testing for Mat Foundation [3]

Concrete consistency values that were less than the allowed limit of class S4 determined on the day of the test by the slump method at the site were corrected by in situ addition of admixture (superplasticizer) in the concrete mixture. The superplasticizer helps to correct the slump to the required consistency class and is suitable for maintaining the flow over a longer period of time (if the concrete mix is on the construction site for a longer period of time, but not longer than one hour), good placing with lower water-cement factors, and the homogeneity of the concrete mixture (at the maximum dosage of 0.15% of the amount of the binder in the concrete). Concrete that had consistency values greater than the permissible limit of the consistency class S4 ascertained on the day of the test by the slump method, was installed, with particular attention being paid to the installation itself, i.e., vibrating. [3]

During the construction of walls of WWTP facilities, the minimum values of concrete consistency for walls, tested by the slump method on November 14, 2015 and December 1, 2016 were 140 mm, which were below the allowed limit of the consistency class S4 (150 mm). The concrete consistency result for walls, measured on November 18, 2015 was 240 mm, while on December, 12, 2015 and February 4 and 13, and March 7, 2016 were 230 mm, which were above the permissible upper limit of the consistency class S4 (220 mm). [3] The values outside the given limits were also corrected in situ, using the same technological approach as was explained for mat foundation.

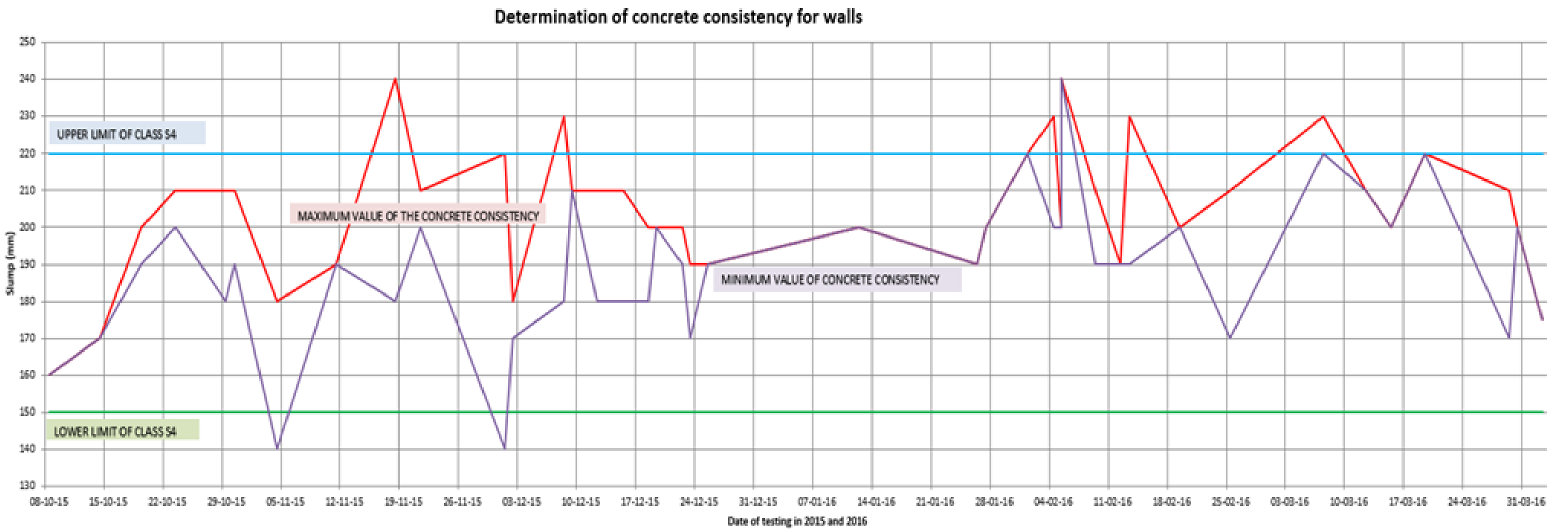


Figure 40 - Test Results of the Concrete Consistency Testing for RC Walls [3]

Water penetration into the concrete under the pressure is another set of tests conducted during construction of facilities of WTTP in Bihać, taking in consideration nature of facilities. During the design phase of concrete structures of Waste Water Treatment Plant, parameters concerning concrete tightness were taken early into consideration, so that all concrete covers were 5 cm thick, what was satisfactory for water tightness of concrete elements considered. Test showed that all maximum depths of water penetration obtained were below the maximum permissible value to which water can penetrate into the concrete. The maximum water penetration value was 26 mm on samples taken during casting of walls and mat foundation for Activated Sludge Tank. [15] The results of water testing on penetration into concrete under pressure are displayed in *Figures 41-43*.

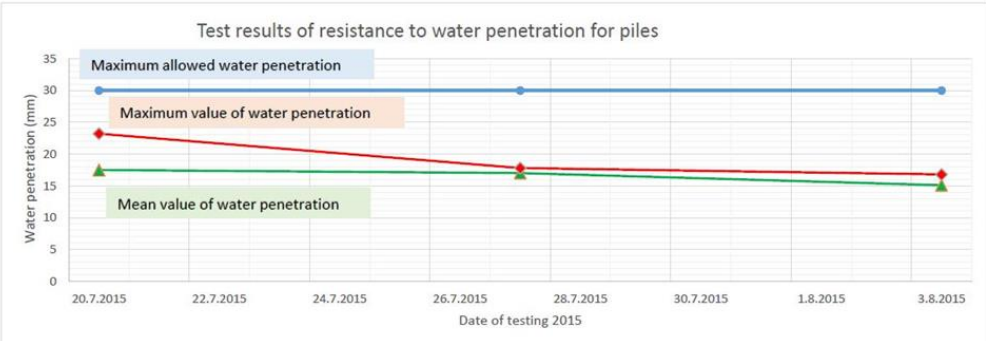


Figure 41 - Test Results of Water Penetration into Concrete under Pressure for Piles [15]

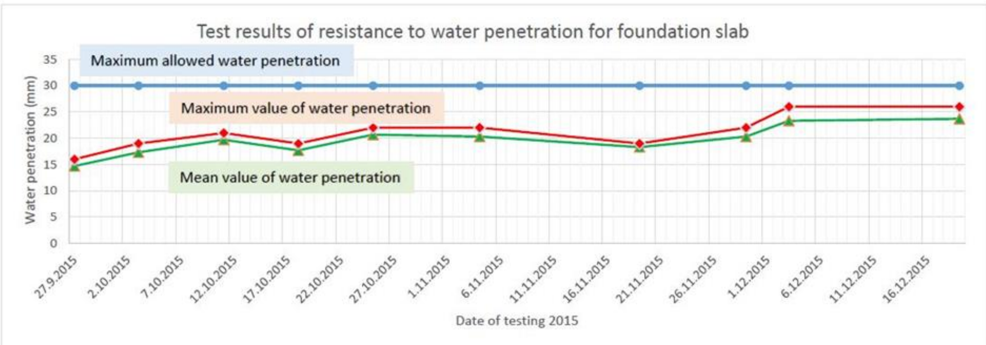


Figure 42 - Test Results of Water Penetration into Concrete under Pressure for Mat Foundation [15]

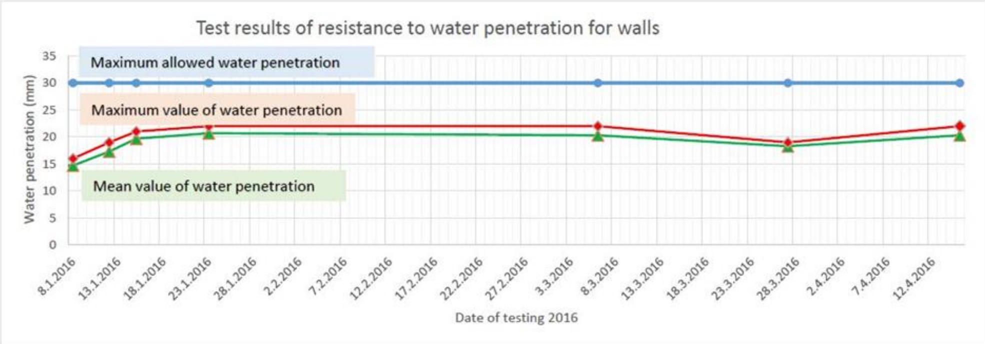


Figure 43 - Test Results of Water Penetration into Concrete under Pressure for Walls [15]

The Rulebook on technical regulations for construction products to be built in concrete structures [42], provision A.2.1.15. states that concrete structures exposed to the environmental impact labelled with exposure classes XF1 and XF3, according to EN 206, meet the durability requirements if testing results fulfil criteria according to standard JUS U.M1.016. So, according to concrete mix design and the Rulebook, testing of concrete frost resistance conducted on concrete samples taken during the construction of the WWTP Bihać was done in accordance with JUS U.M1.016. Conformity criteria requires less than 25% of concrete strength change during the destructive method treatment, so the results shown in *Figure 44* confirm that RC concrete structural elements of different hydrotechnical facilities fulfill the requirements of the standard. [16]

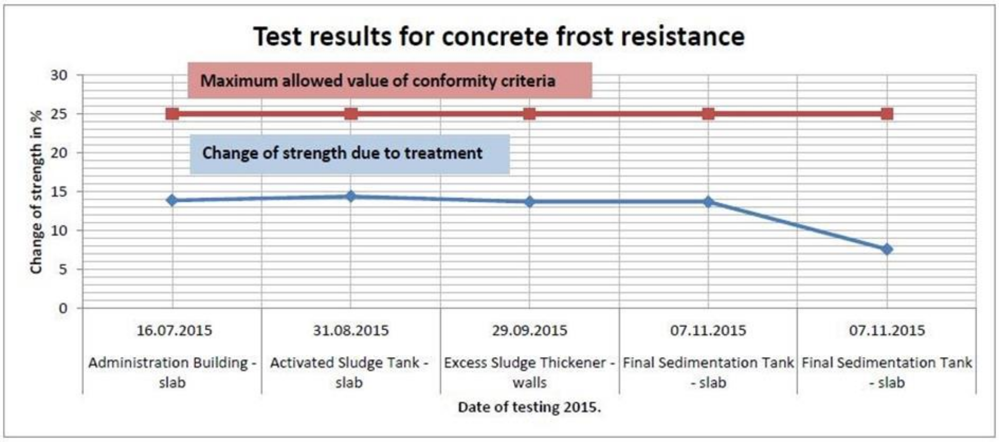


Figure 44 - Test Results for Freeze-Thaw Resistance of Concrete [16]

As mentioned earlier, the reinforcement for RC structural elements for the WWTP facilities was made of the reinforcing steel ribbed bars with characteristic yield stress of 500 N/mm^2. All reinforcement was clean and free from pit corrosion, loose rust, mill scale, paint, oil, grease, adhering earth, or any other material that might impair the bond between the concrete and the reinforcement, or which could may cause corrosion of the reinforcement, or might be detrimental to the quality of concrete. Reinforcement was stored on racks or supported clear of the ground. Different types and sizes of reinforcement were kept separate. Reinforcement test samples were taken continuously during the construction, complying with the quality control program. Testing results of tensile strength for reinforcing bars Ø8, Ø10, Ø14, Ø16 mm and reinforcing mesh Q335 are presented in *Figures 45 – 49*.

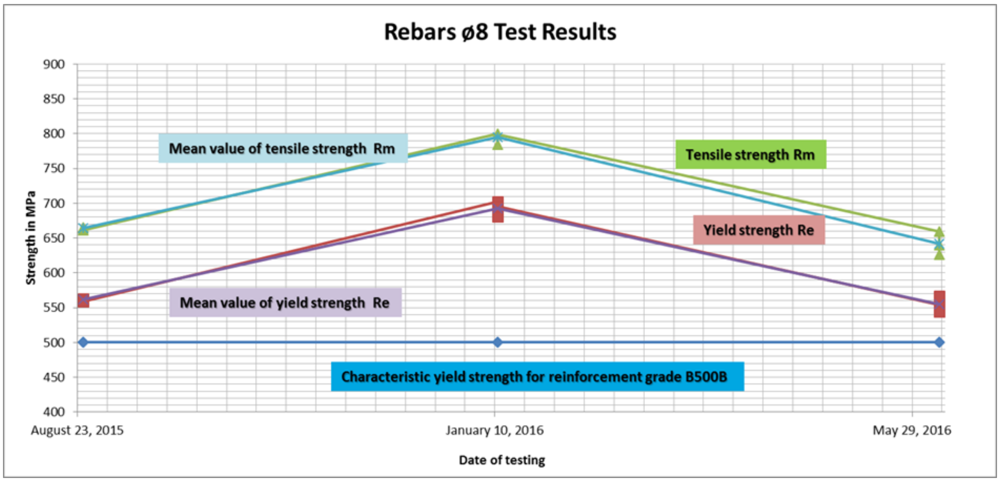


Figure 45 - Test Results for Rebar Ø8 [5]

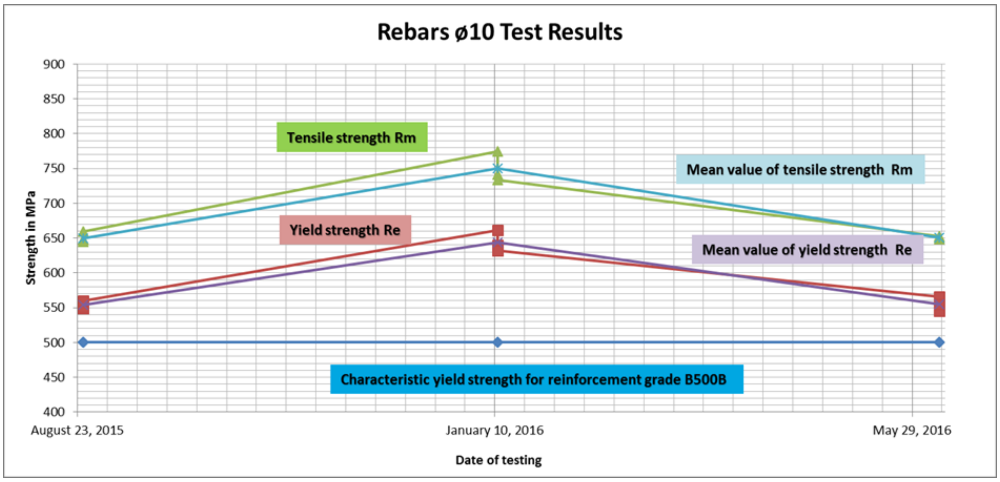


Figure 46 - Test Results for Rebar Ø10 [5]

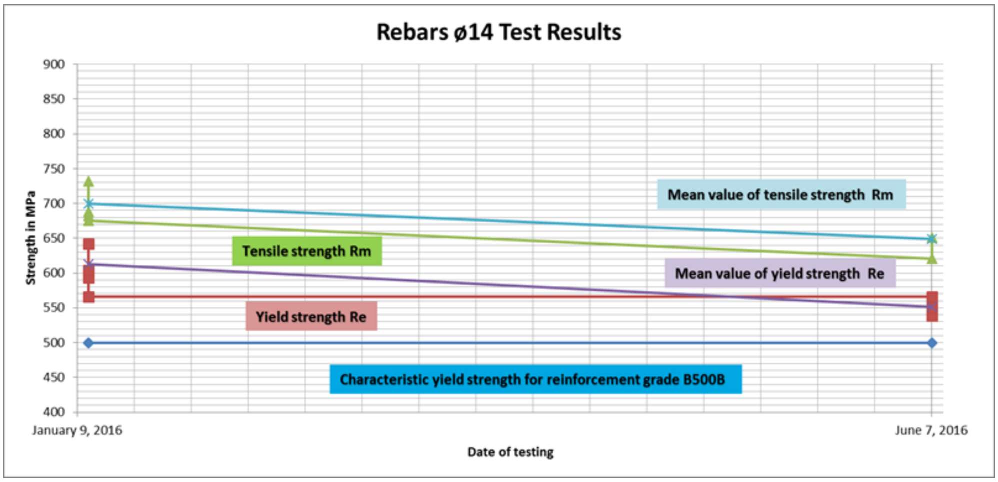


Figure 47 - Test Results for Rebar Ø14 [5]

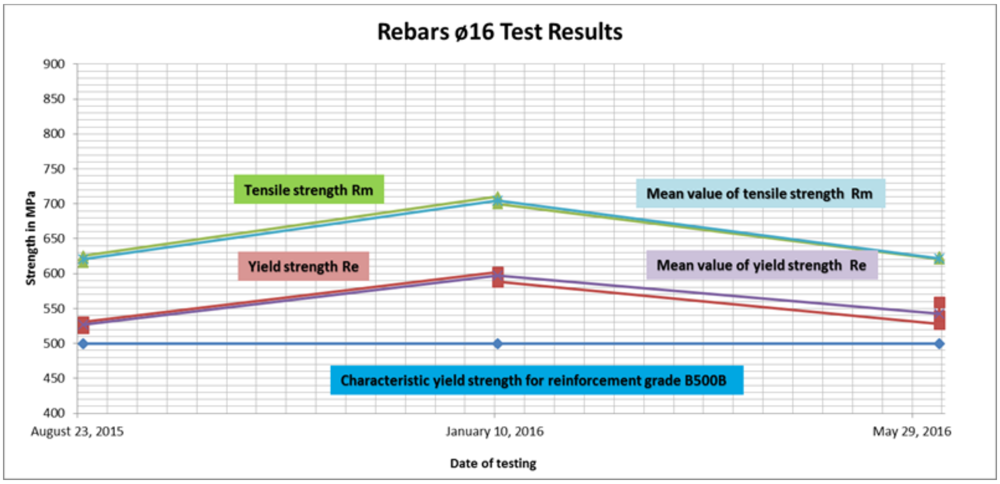


Figure 48 - Test Results for Rebar Ø16 [5]

The test results showed that yield strength values for bars Ø8, Ø10, Ø14, Ø16 mm and reinforcing mashes Q335 met the required limiting value for reinforcement B500B, which is ≥ 500 MPa. The yield strength test results values for bars Ø19, Ø20, Ø22 and Ø25, which were not shown by the graph here, (since only one test sample was required according to quality control program), also met the required limiting value for reinforcement B500B, which is ≥ 500 MPa. [5]

The shear strength value of the weld for reinforcing mesh Q335, tested on August 24, 2015 are 12,6 and 13,2 kN (for two samples), was above the limit value of 6,3 kN . Shear strength values of the welds for reinforcing mesh Q335, tested on June 6, 2016 were 12,8 and 13,6 kN (for two samples), which were also above the limit value. [5]

Tensile strength for all rebars satisfied the R_m/R_e ratio, since all ratio values were above the limit value of 1.08. [5]

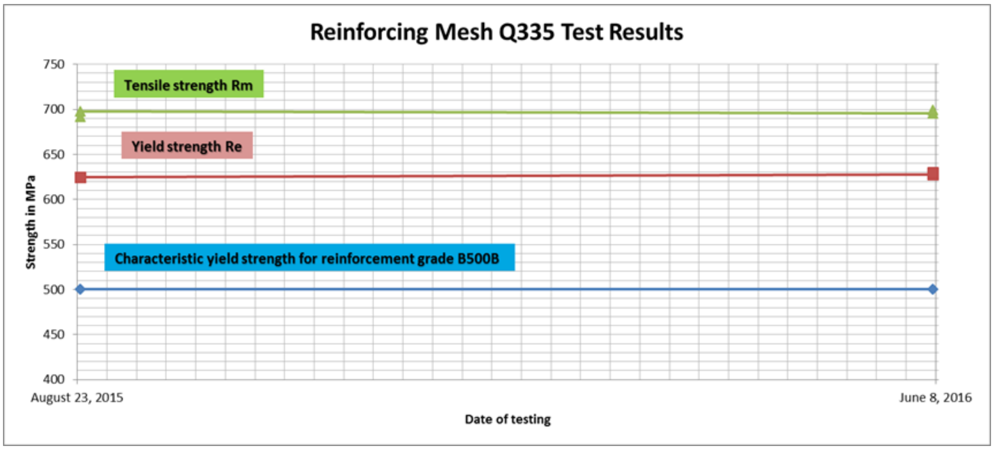


Figure 49 - Test Results for Reinforcing Mesh Q335 [5]

CONCLUSION

Quality control of reinforced concrete structures is a time demanding process that does not end with concrete placement; on the contrary, it lasts for some period after concrete placement since concrete develops its designed strength after at least 28 days. Considering this fact, quality control of concrete requires adequate quality control plan that encounters both quality control of fresh and hardened concrete.

Depending on concrete characteristics defined in the quality control plan, all test procedures which would confirm the concrete quality compliance to the concrete mix design should be predicted.

Considering concrete characteristics and behavior of structural element, and according to the quality control plan, concrete has to comply with two types of tests – conformity testing and identity testing. Conformity testing is the testing conducted in order to verify the concrete mix design recipe and to confirm expected behavior of concrete according to the mix design. This type of testing is performed in a concrete factory.

Identity testing is testing conducted in situ or in laboratory, which verifies that concrete delivered to the construction site is of the same characteristics as the one produced in the concrete factory. Identity testing relates to set of various test procedures depending on the concrete characteristic tested.

In addition to this, the procedure of conducting conformity and identity testing was presented through the concrete control quality on the construction site of the Waste Water Treatment Plant in Bihac and was done during the actual construction. It is also possible to overview timeline of testing from the figures from the last chapter. This timeline supports the fact that in order to gain overall control quality insight, a significant number of samples has to be taken over a longer period of time, preferably in all weather conditions occurring.

The conformity and identity testing conducted during the construction of the WWTP showed positive evaluation in all aspects of concrete control quality. Starting from the testing of compressive strength, the results showed that all samples met both criteria – conformity and identity, in spite of some significant oscillations in the test results. These oscillations may be a consequence of concrete manufacturing technology, i.e., in dosing of concrete constituents, storing the cement and aggregates and their treatment in the cold weather conditions.

When analyzing aspects of concrete consistency identity testing, it was concluded the concrete consistency also met the requirements defined by the European standards.

There were no major problems with obtaining the adequate consistency of concrete during its placement since there was always admixture available in the liquid form on the construction site, should the intervention on concrete consistency be required. As stated in earlier considerations, all deviations in concrete consistency were easily solvable with addition of plasticizer.

Concrete resistance to water penetration is generally, as all other parameters, defined in the phase of design of the concrete mixture. Value allowed for water penetration is always lower than the thickness of the concrete cover to reinforcement. Test results showed that the maximum water penetration value was 26 mm, which was quite appropriate, since the limit was 30 mm, while the concrete cover to reinforcement in all water containing facilities was 5 cm.

When analyzing the test results of concrete frost resistance, the analysis is made according to the Rulebook on technical regulations for construction products to be built in concrete structures [42]. The Rulebook states that the frost resistance is tested according to the national standard JUS U.M1.016 if the concrete is of exposure classes XF1 and XF3. If the concrete is of exposure classes XF2 and XF4, it has to be tested under the terms of BAS CEN/TS 12390-9:2007. Since all concrete structural elements were of exposure class XF3, the conducted tests were in accordance with national standard, where all tested samples met the standard requirements.

Taking into consideration that all facilities were constructed of reinforced concrete, there was also a need for testing of reinforcement as integral part of reinforced concrete elements. These tests were conducted according to EN 10002-1:2002, where all requirements were also met.

To summarize from this study and its supporting test results, it can be concluded that a lot of effort and attention was dedicated to concrete control quality on the construction site of the Waste Water Treatment Plant in Bihać. By analyzing test results, it can be stated that quality assurance of concrete was successfully conducted. Such approach gained experience and lessons learned during the construction of this facility can serve as a model approach for quality control of concrete during construction of other concrete facilities and structures in Bosnia and Herzegovina, in accordance to European standards.

REFERENCES

[1] ArcelorMittal Zagreb d.d.o, 2014. "Tehnička uputa za čelik za armiranje betona B500B u šipkama", ArcelorMittal Zagreb, Draškovićeva 49, Zagreb, Hrvatska;

[2] Bajramović E, Džidić S, 2017. "Accredited Laboratories in Process of Introducing Construction Products to EU Market", Book of Proceedings of the 3rd International Scientific and Professional Conference "SFERA 2017-Concrete Technologies", March 23-24, 2017, Mostar, Bosnia and Herzegovina, pp 100-105;

[3] Čehić E, Kozlica S, Džidić S, 2016. "Concrete Consistency Testing during Construction of the Waste Water Collection and Treatment Plant in Bihać according to EN 206:2014", Book of Proceedings of the 12th International Scientific and Professional Conference on Contemporary and Theory and Practice in Construction, University of Banja Luka Faculty of Architecture, Civil Engineering and Geodesy, December 7-8, 2016, ISBN 978-99976-663-3-8, UDK 69:71/72(082)(0.034.2) 624(082)(0.034.2), COBISS.RS-ID 6340024, Banja Luka, Bosnia and Herzegovina, pp 503-510;

[4] Džidić S, Kozlica S, Ćehić E, 2016. "Experiences in Testing of Compressive Strength of Concrete during Construction of Waste Water Collection and Treatment Plant in Bihać according to BAS EN 206:2014", Book of Proceedings of the 12th International Scientific and Professional Conference on Contemporary and Theory and Practice in Construction, University of Banja Luka Faculty of Architecture, Civil Engineering and Geodesy, December 7-8, 2016, ISBN 978-99976-663-3-8, UDK 69:71/72(082)(0.034.2) 624(082)(0.034.2), COBISS.RS-ID 6340024, Banja Luka, Bosnia and Herzegovina, pp 495-502;

[5] Džidić S, Čehić E, Kozlica S, 2017. "Experience in Testing of Concrete Reinforcement according to EN 10002-1", Book of Proceedings of the 5th International Conference "Contemporary Achievements in Civil Engineering", April 21, 2017, ISBN 978-86-80297-68-2, UDK: 666,98, DOI:10.14415/konferencijaGFS2017.046, Subotica, Serbia, pp 441-449;

[6] Džidić S, Okugić H, Bajramović E, 2017, "Studije o betonu 2016-2017; International BURCH University Sarajevo, CIP 624.012.4, ISBN 978-9958-834-57-8, COBISS.BH-ID 24495622, Sarajevo, Bosnia and Herzegovina;

[7] IGMAT d.o.o., Sarajevo, 2015. “Concrete Mixture Design for Piles Execution on Facilities of Waste Water Treatment Plant in Bihać”, Sarajevo, Bosnia and Herzegovina;

[8] JV Fichtner Water & Transportation GmbH /Una Consulting d.o.o, 2012. “Tender Documentation for the Construction of WWTP in Bihać”, Bihać, Bosnia and Herzegovina;

[9] JV Pfeiffer–Emit, Bauer, 2014. “Method Statement for the Piles Execution of WWTP Bihać”, Bihać, Bosnia and Herzegovina;

[10] JV Pfeiffer – Emit, 2015. “QA/QC Plan for the Construction of WWTP in Bihać”, Bihać, Bosnia and Herzegovina;

[11] JV Pfeiffer – Emit, 2015. “Operation and Maintenance Manual of WWTP”, Bihać, Bosnia and Herzegovina;

[12] JV Pfeiffer-Emit & Zavod za Vodoprivredu d.d. Sarajevo, 2014. “General Documentation of the Final Design of WWTP Bihać”, Bihać, Bosnia and Herzegovina;

[13] JV Pfeiffer–Emit, IGH Mostar, 2015. “Concrete Mix Design for Design and Execution of the Waste Water Treatment Plant in Bihać”, BMZ ID 2007 65 925, Bihać, Bosnia and Herzegovina;

[14] JV Pfeiffer–Emit, 2015/16. “QA/QC Monthly Reports during the Construction of WWTP in Bihać”, Bihać, Bosnia and Herzegovina;

[15] Kozlica S, Čehić E, Džidić S, 2017. “Experience in Testing of Concrete for Water Penetration during Construction of WWTP in Bihać”, Book of Proceedings of the 5th International Conference “Contemporary Achievements in Civil Engineering”, April 21, 2017, , ISBN 978-86-80297-68-2, UDK: 666.972.52, DOI:10.14415/konferencijaGFS2017.045, Subotica, Serbia, pp 433-440;

[16] Kozlica S, Čehić, E, 2017. “Experiences in Testing of Concrete Frost Resistance during the Construction of WWTP in Bihać“, Book of Proceedings of the 11th International Scientific Conference on Production Engineering - Development and Modernization of Production – RIM 2017“, October 4-7, 2017, Sarajevo, Bosnia and Herzegovina, pp 411-416;

[17] Li Z, 2011. ”Advanced Concrete Technology”, John Wiley and Sons Inc., ISBN 978-0-470-43743-8, New Jersey, USA;

[18] Putzmeister, 2011. “Concrete Technology for Concrete pumps”, Offizin Scheufele GmbH & Co. KG, Germany;

[19] Radić, J i suradnici, 2006. “Betonske konstrukcije – Priručnik”, Hrvatska sveučilišna naklada, Građevinski fakultet Sveučilišta u Zagrebu, ISBN 953-169-126-6, Zagreb, Hrvatska;

[20] Sika, 2005. “Sika – Handbook on Concrete”, Sika Services AG, Corporate Construction, CH-8048 Zürich, Switzerland.

STANDARDS AND NORMS

[21] BAS CEN/TS 12390-9:2007 Testing hardened concrete - Part 9: Freeze-thaw resistance – Scaling, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[22] BAS EN 10002-1:2002 – Metallic materials – Tensile testing – Part 1: Method of test at ambient temperature, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[23] BAS EN 10027-1:2007 – Designation systems for steels – Part 1: Steel names, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[24] BAS EN 1008:2002 - Mixing water for concrete - Specification for sampling, testing and assessing the suitability of water, including water recovered from processes in the concrete industry, as mixing water for concrete

[25] BAS EN 10080:2007 Steel for the reinforcement of concrete - Weldable reinforcing steel – General, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[26] BAS EN 12350-1:2010 - Testing fresh concrete - Part 1: Sampling, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[27] BAS EN 12350-2:2010, Testing fresh concrete – Part 2: Slump – test, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[28] BAS EN 12350-5:2010 - Testing fresh concrete – Part 5: Flow table test, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[29] BAS EN 12390-1:2013, Testing hardened concrete - Part 1: Shape, dimensions and other requirements for specimens and moulds, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[30] BAS EN 12390-2:2010, Testing hardened concrete - Part 2: Making and curing specimens for strength tests, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[31] BAS EN 12390-3:2010, Testing hardened concrete - Part 3: Compressive strength of test specimens, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[32] BAS EN 12390-4:2003, Testing hardened concrete - Part 4: Compressive strength - Specification for testing machines, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[33] BAS EN 12350-7:2010 - Testing fresh concrete – Part 7: Air-content – Pressure methods, Specification for testing machines, Institut za standardizaciju BiH, Istočno Sarajevo, Bosna i Hercegovina;

[34] BAS EN 12390-8:2010 - Testing hardened concrete - Part 8: Depth of penetration of water under pressure, Institut za stadardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[35] BAS EN 12620+A1:2009 - Aggregates for concrete, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[36] BAS EN 13670:2011 - Execution of concrete structures, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[37] BAS EN 196 - Methods of testing cement, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[38] BAS EN 197 – Cement, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[39] BAS EN 206:2014 – Concrete - Specification, performance, production and conformity, Institut za standardizaciju Bosne i Hercegovine, Istočno Sarajevo, Bosna i Hercegovina;

[40] JUS U.M1.016 – Ispitivanje betona prema dejstvu mraza 1992;

[41] JUS U.M1.032 - Beton - Mjerenje temperature svježeg betona, 1981;

[42] Pravilnik o tehničkim propisima za građevinske proizvode koji se ugrađuju u betonske konstrukcije, Službene novine FBiH br. 2/06, 72/07, 32/08.

INTERNET RESOURCES

[43] http://djuzelic.ba/djelatnosti/proizvodnja-betona/

[44] http://fidic.org/

[45] http://fidic.org/sites/default/files/FIDIC_Suite_of_Contracts_0.pdf

[46] http://tpm.fsv.cvut.cz/student/documents/files/BUM1/Steel

[47] http://www.cedd.gov.hk/eng/publications/standards_handbooks_cost/doc/stan_cs1/CS1-2010%20Vol%202%20Rev%2000-101130.pdf

[48] http://www.cemex.co.uk/Userfiles/datasheets/mortar-cementitious.pdf

[49] http://www.everything-about-concrete.com/images/s-tube-valve-system.jpg

[50] http://www.nexe.hr/userDocsImages/dokumenti/TECHNICAL%20INSTRUCTION%20FOR%20CEMENT%20CEM%20III%20B%2032,5N%20SR-LH.pdf

[51] http://slideplayer.com/slide/4901592/16/images/17/Workability+&+Consistency.jpg

[52] http://www.voda.hr/sites/default/files/dokumenti/poglavlje_6-armiracki_radovi.pdf

[53] https://3qcake159kca33qhh23so84g-wpengine.netdna-ssl.com/wp-content/uploads/2014/09/Air-Content-2013_2.pdf

[54] https://cdn3.volusion.com/vfhqd.bkyvh/v/vspfiles/photos/ACM-6-3.jpg?1414127397

[55] https://connect.ncdot.gov/resources/Materials/PublishingImages/air_pot_16.jpg

[56] https://www.inconcreto.net/2888-concrete-quality-control-planning-according-to-uni-en-13670

[57] https://www.scribd.com/doc/290544655/BS-en-12350-2-2009-Testing-Fresh-Concrete-Slump-test